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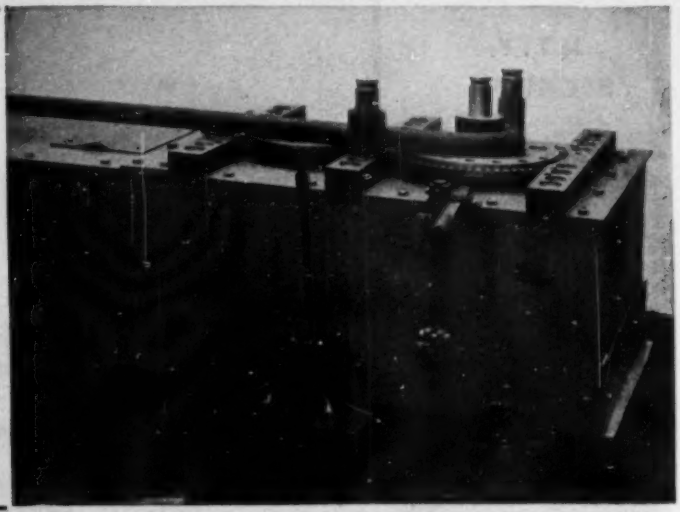

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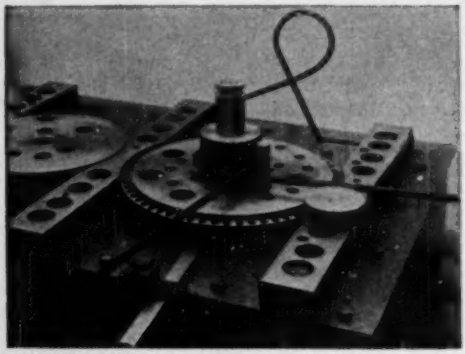

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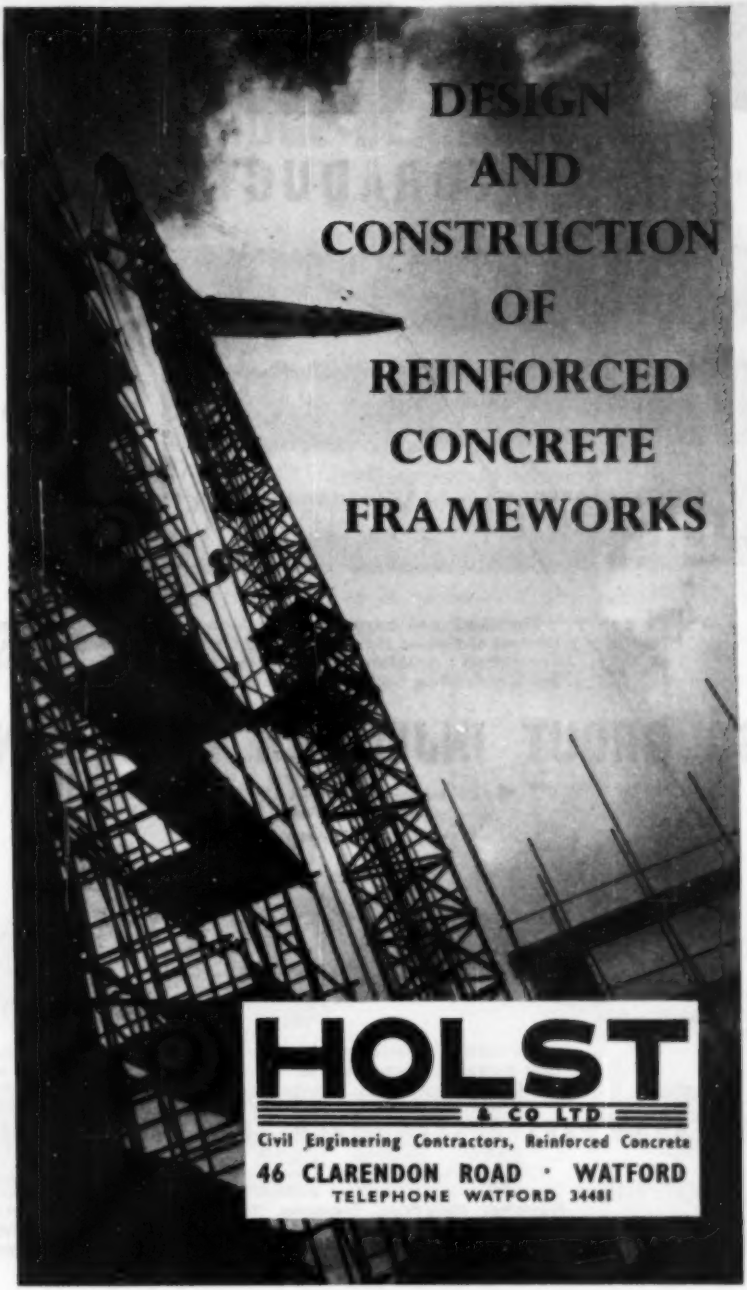
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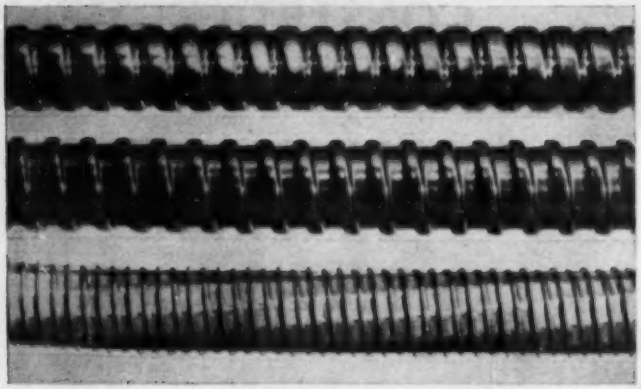

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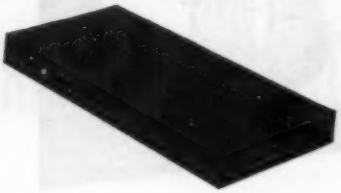

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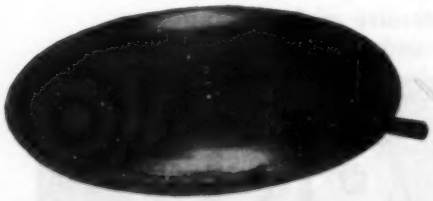

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27	10·6	64
30	11·8	81
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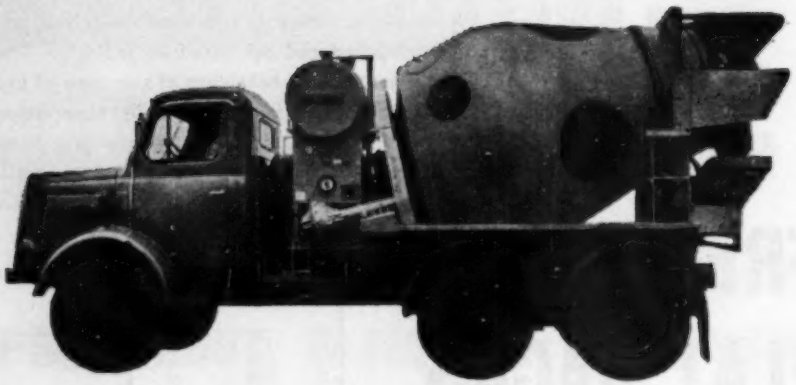

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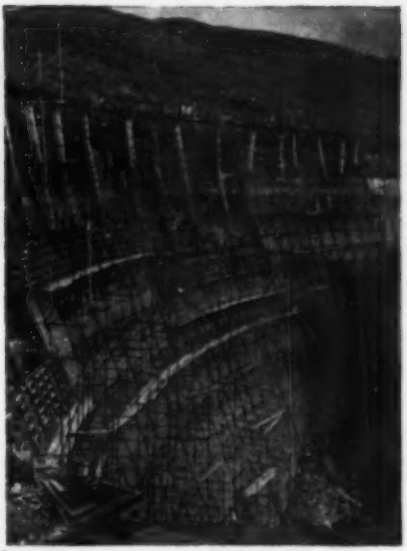

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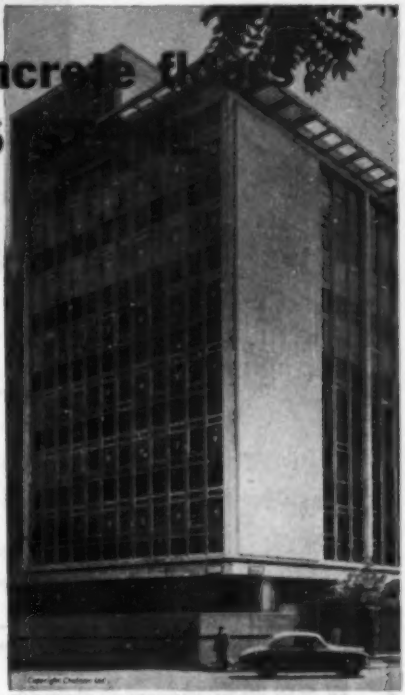

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Consulting Engineer: John DeBromhead & Partners
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Our skilled staff are always ready to assist you in the initial design stages or they are available to carry out the complete design work to the architect's specification. Complete sub-contracts can be undertaken. For further information about the concrete floors write to us

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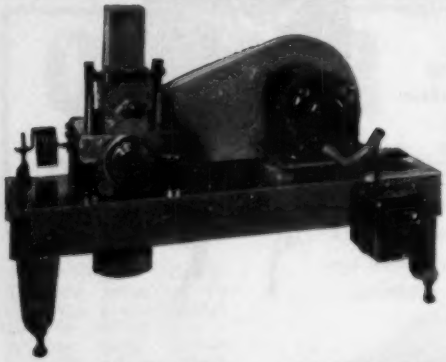

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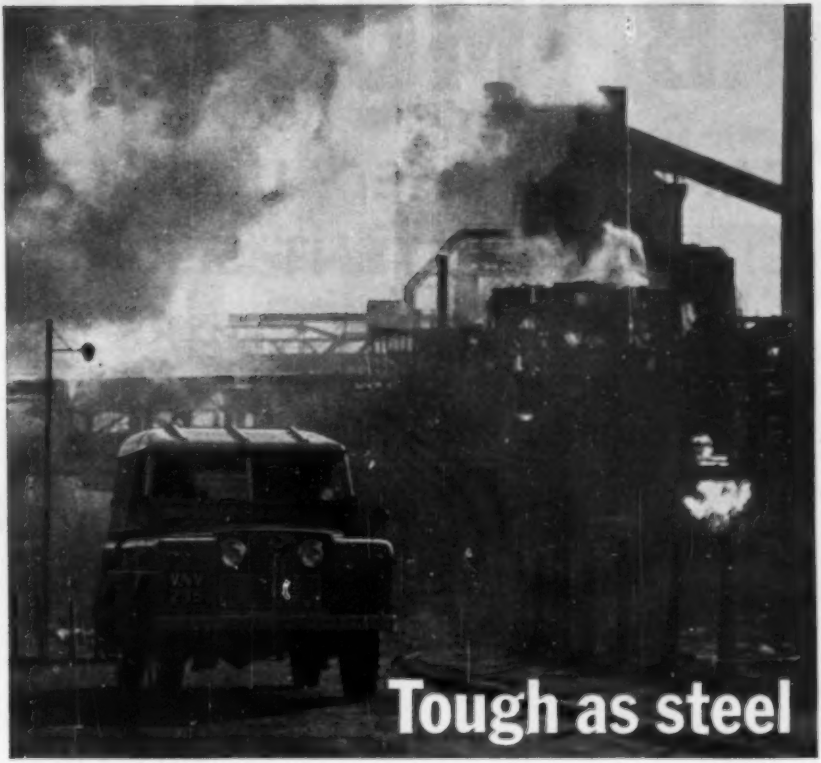

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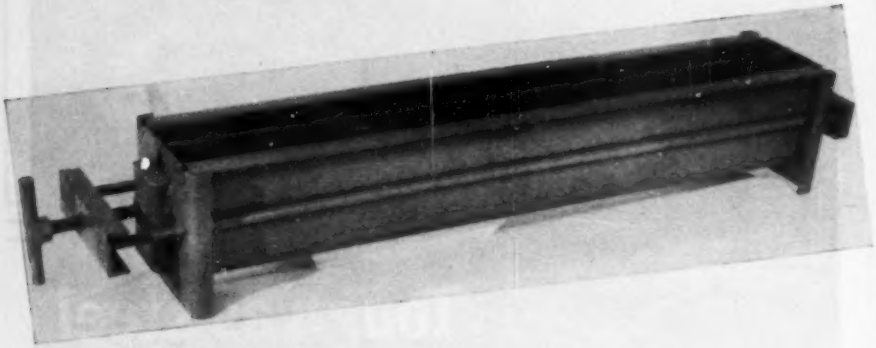

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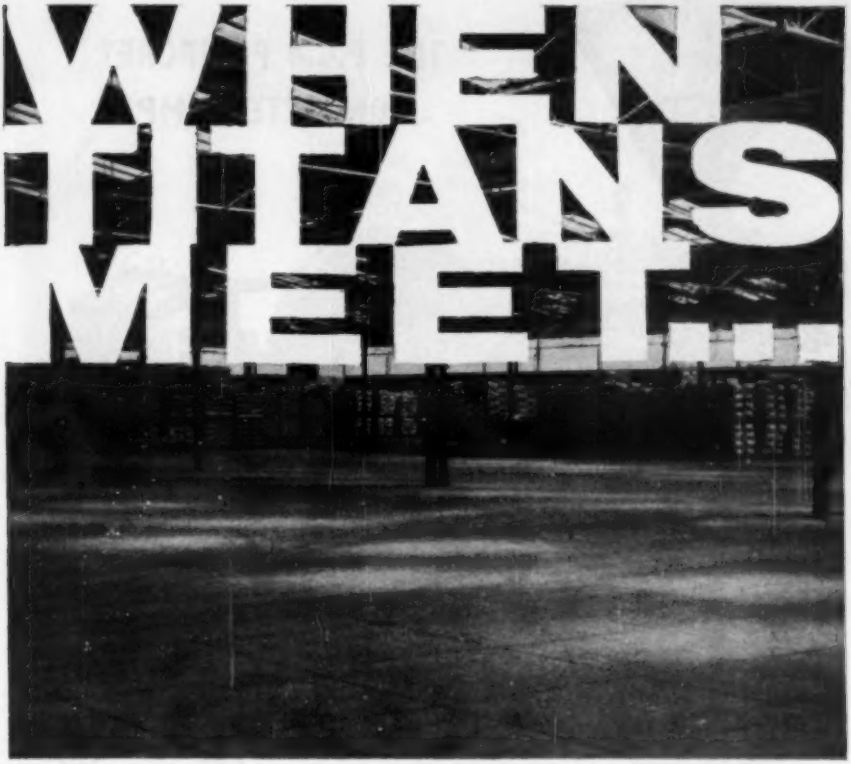

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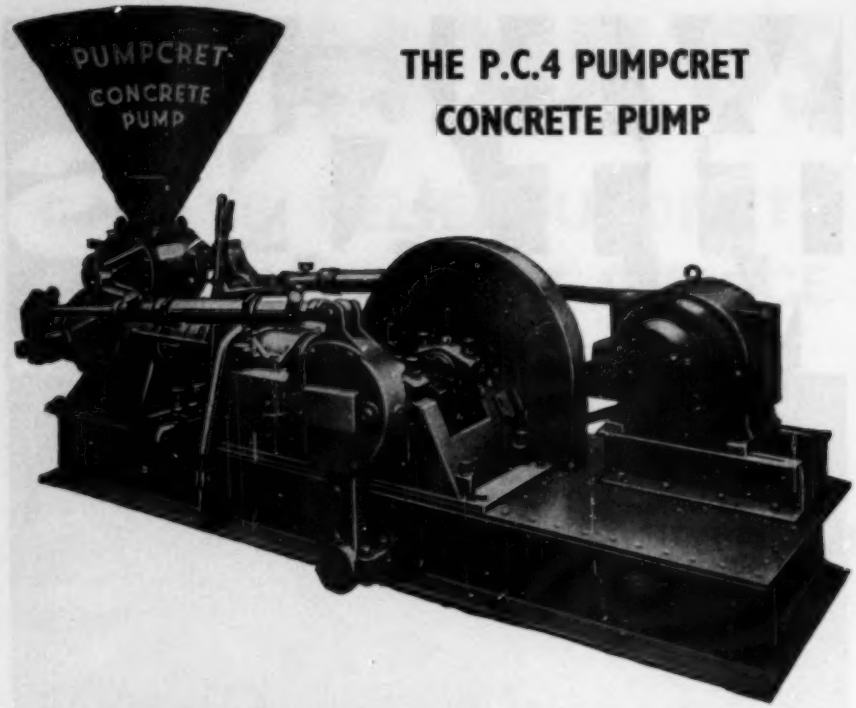

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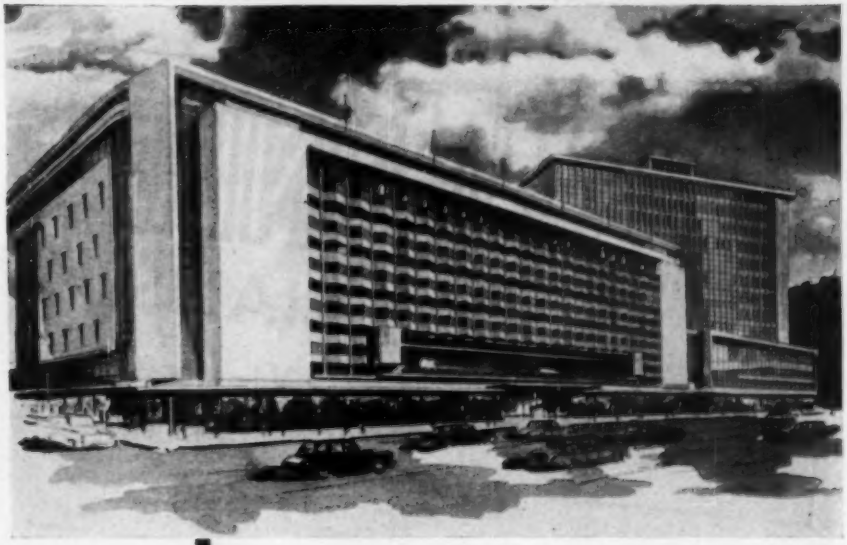

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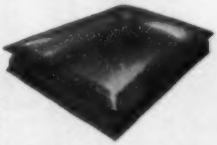

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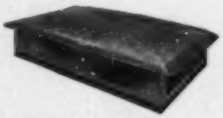

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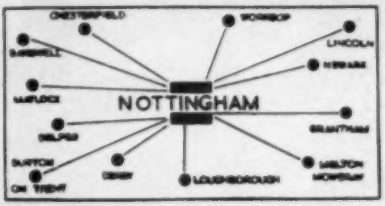

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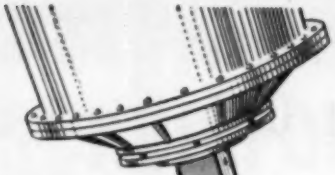

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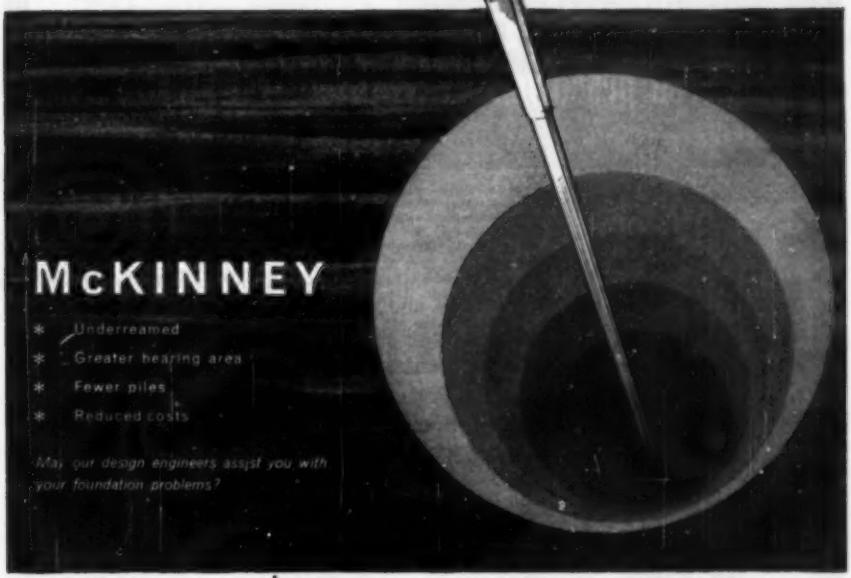

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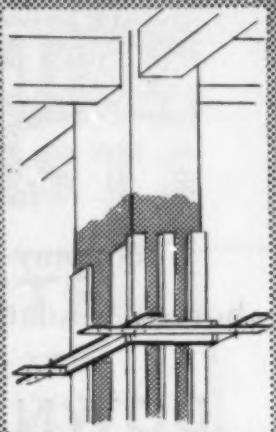

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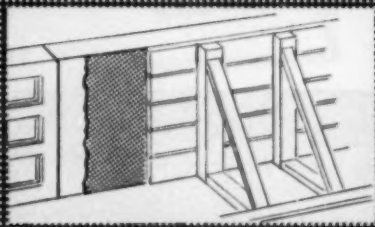

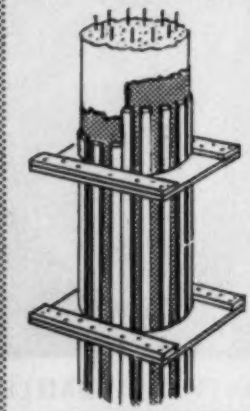

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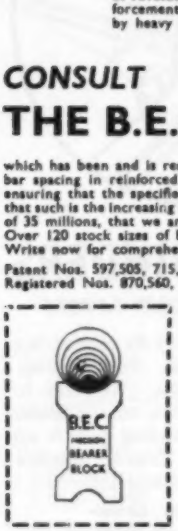

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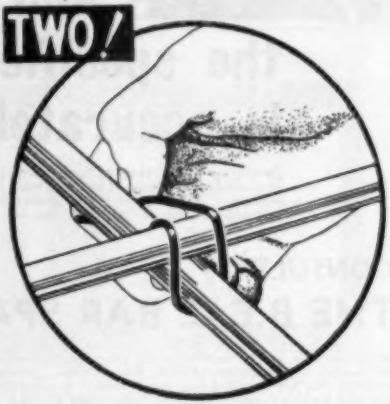

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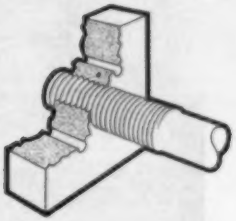

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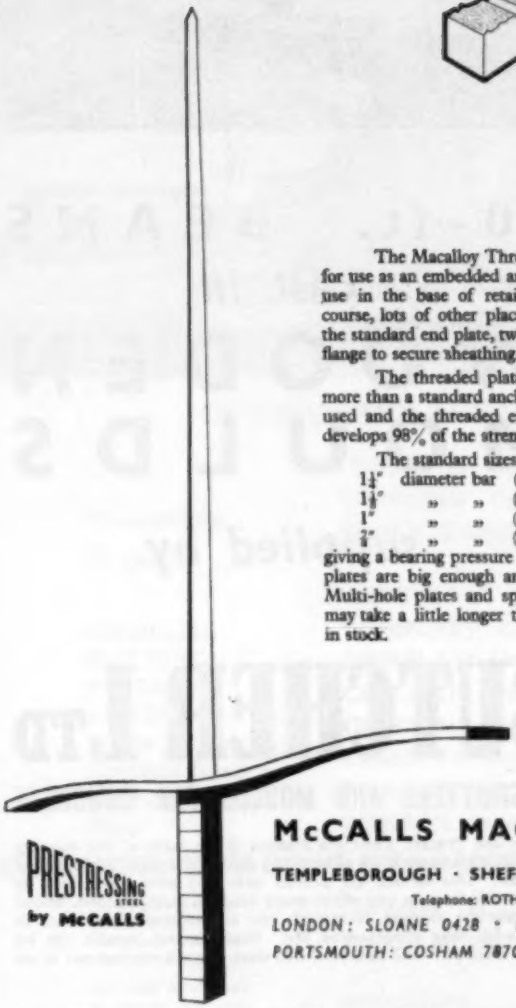

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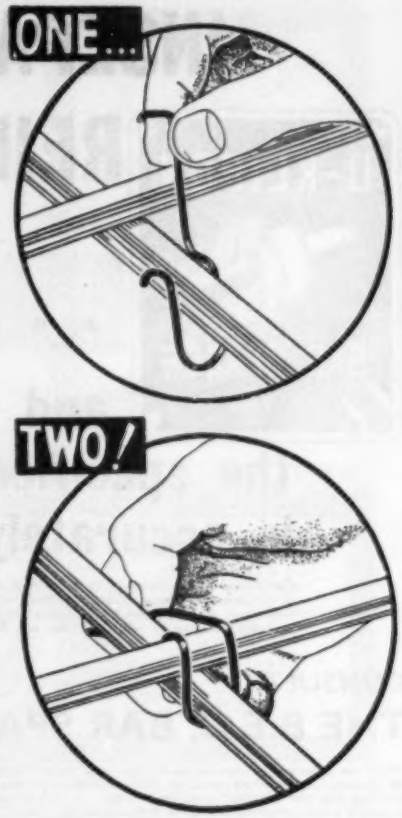

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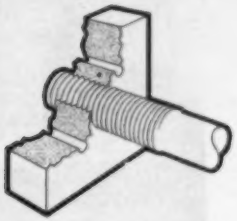

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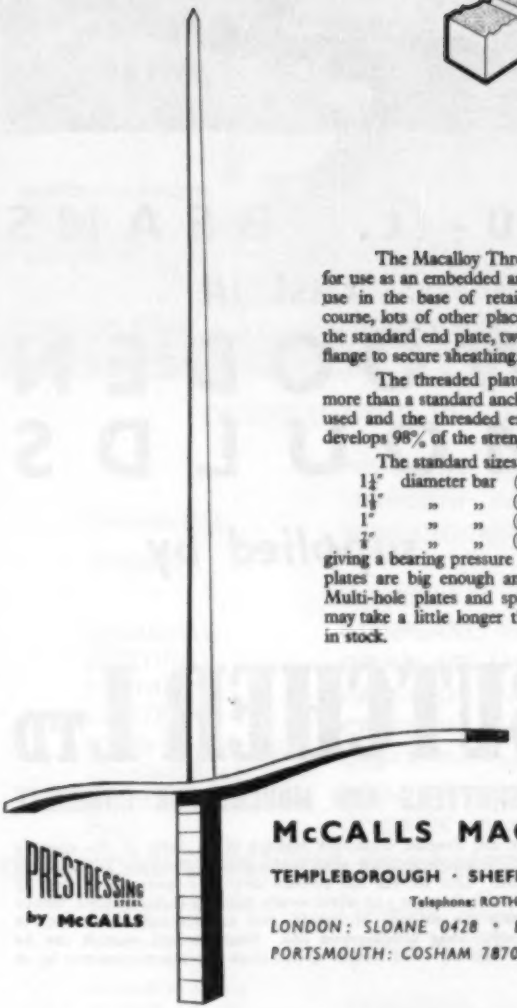

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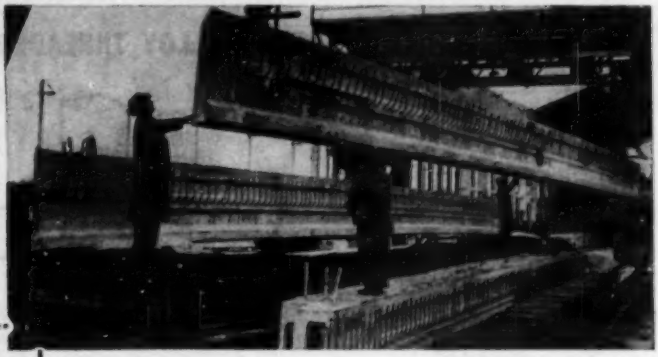

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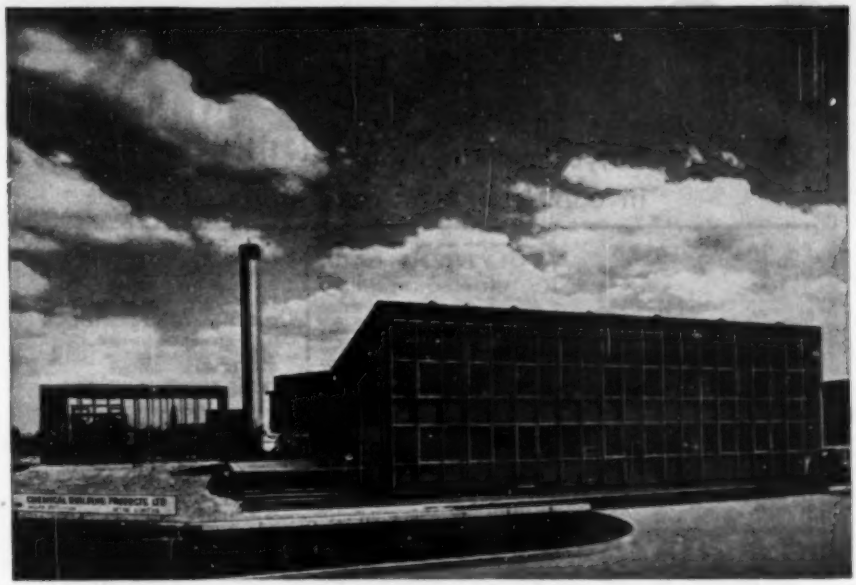

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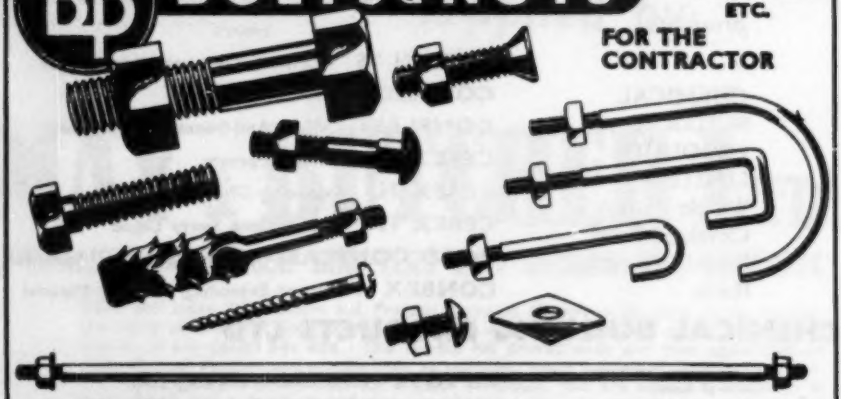

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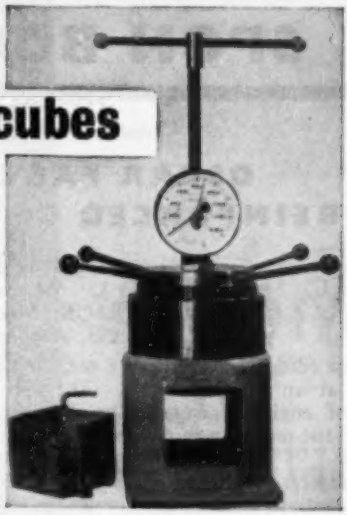

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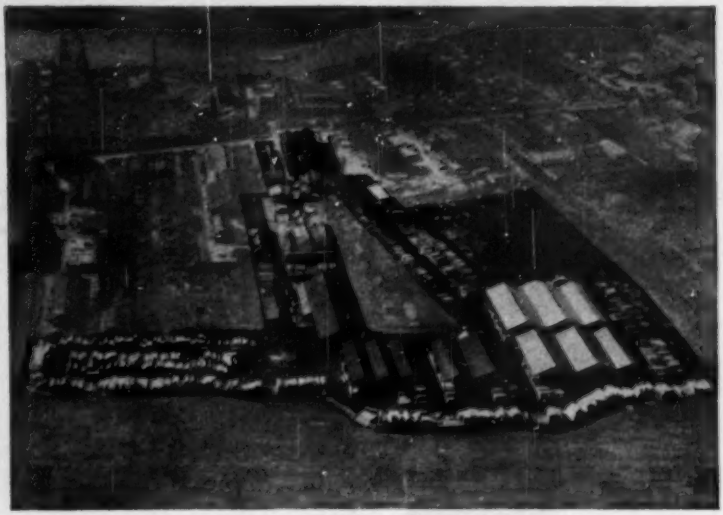

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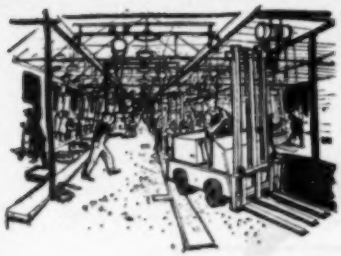

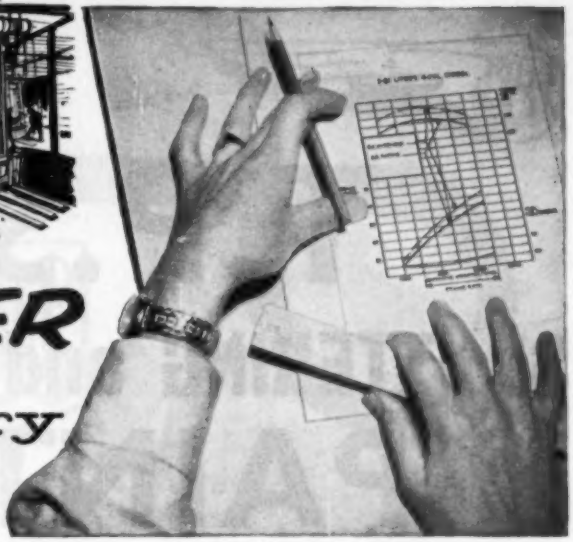

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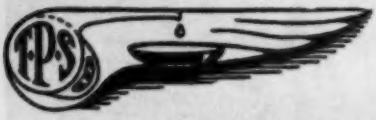

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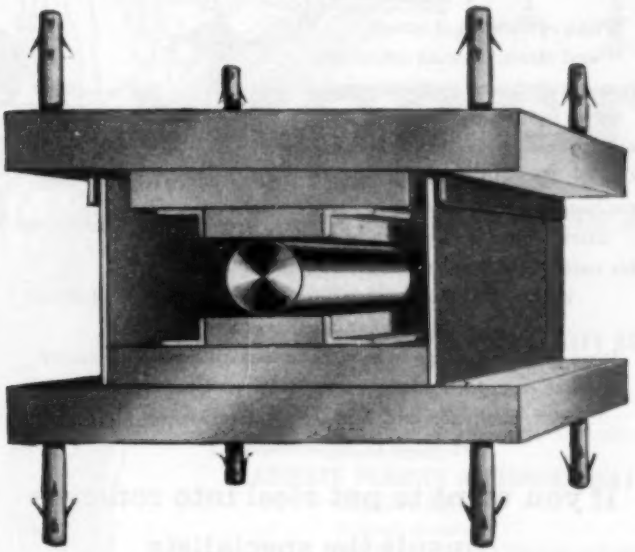

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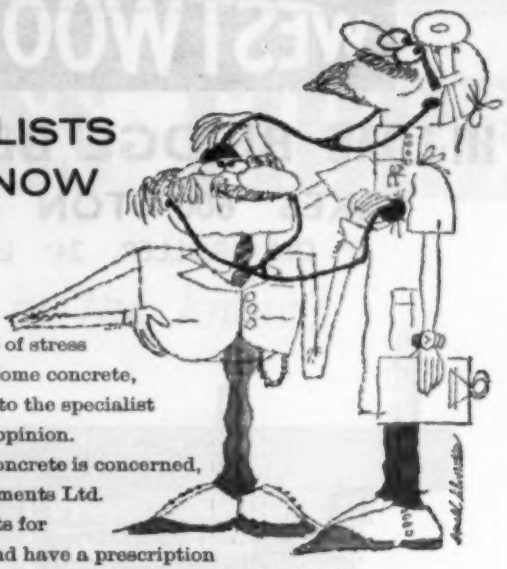

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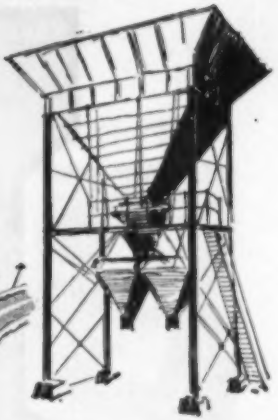

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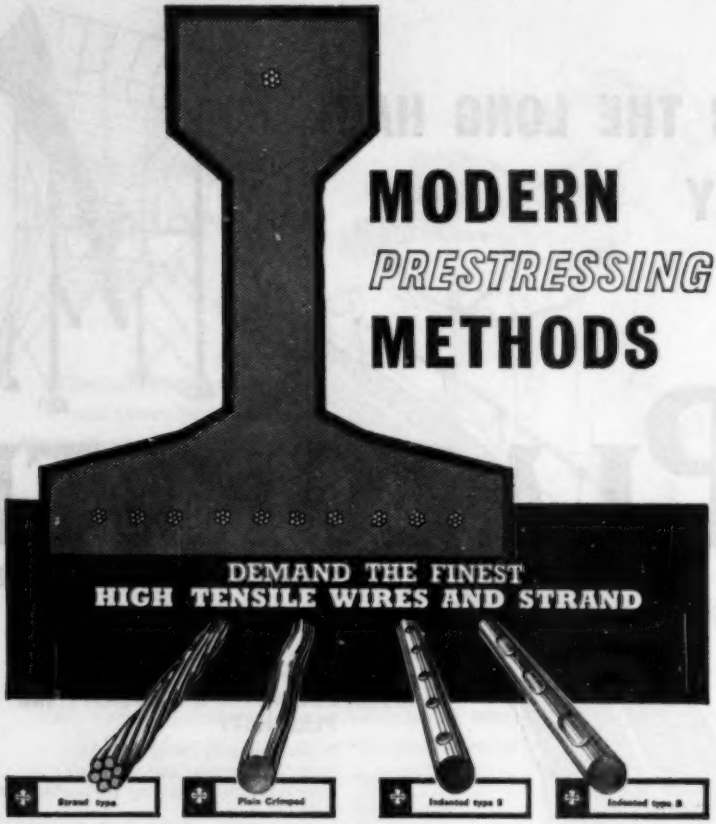

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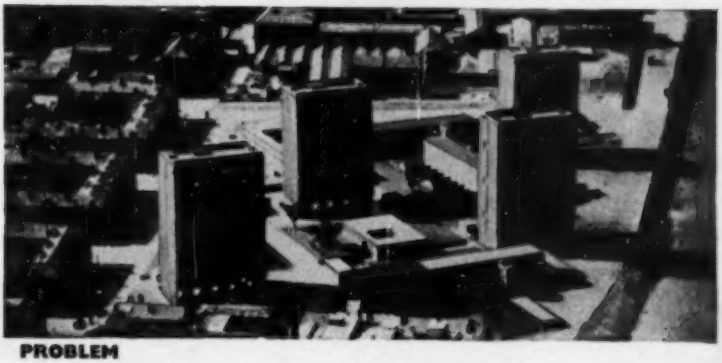
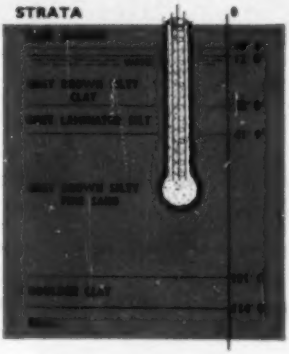


PROBLEM

Four seventeen-storey blocks of flats were constructed on this site, the floor area of each being about 90′ x 90′, and imposing a load at ground level of about thirty cwts. per square foot. As will be seen from the strata details there is a very high water table about 12′ below ground level, and between that and the rock, which is over 100′ down, there is nothing but silt and sand. Ordinary footings would have been out of the question due to the high water table, and the problem was how to carry this high concentrated load without taking piles right down to the rock.

STRATA



SOLUTION

Trial penetration tests with the full size machine were taken, and it was found by this means that Franki piles with their enlarged bases could be formed in the silty sand at about 55′ to 57′, carrying working loads of 45 tons per pile. The alternative would have been to have taken piles down to the rock. All the piling was eventually carried out to an average depth of 57′ founded in the silty sand, the piles being reinforced with six ⅞″ dia. rods with helical binding at 6″ pitch.

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General Contractors: A. A. Stuart & Sons (Glasgow) Ltd.
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Working Load: 45 tons · **Average Length:** 57 feet

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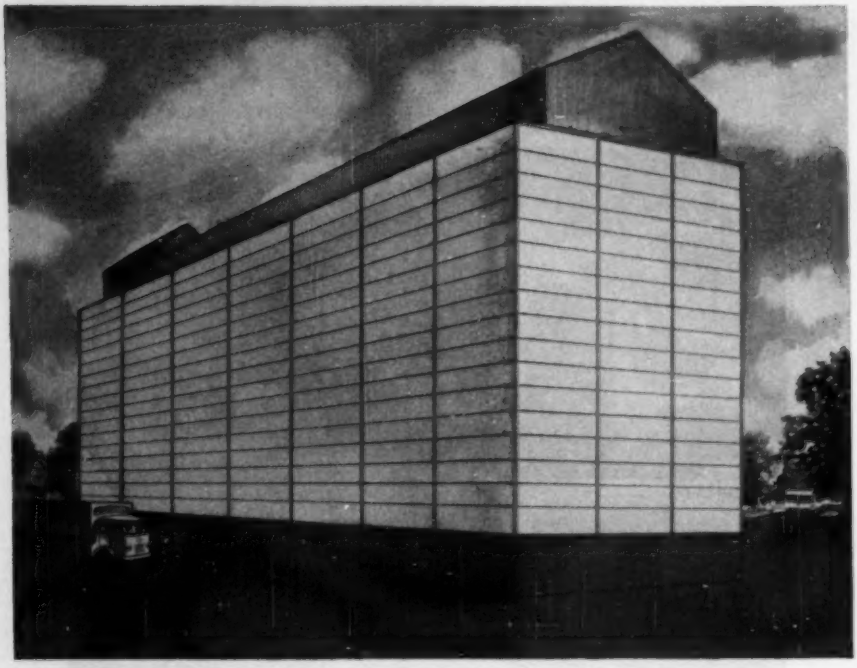

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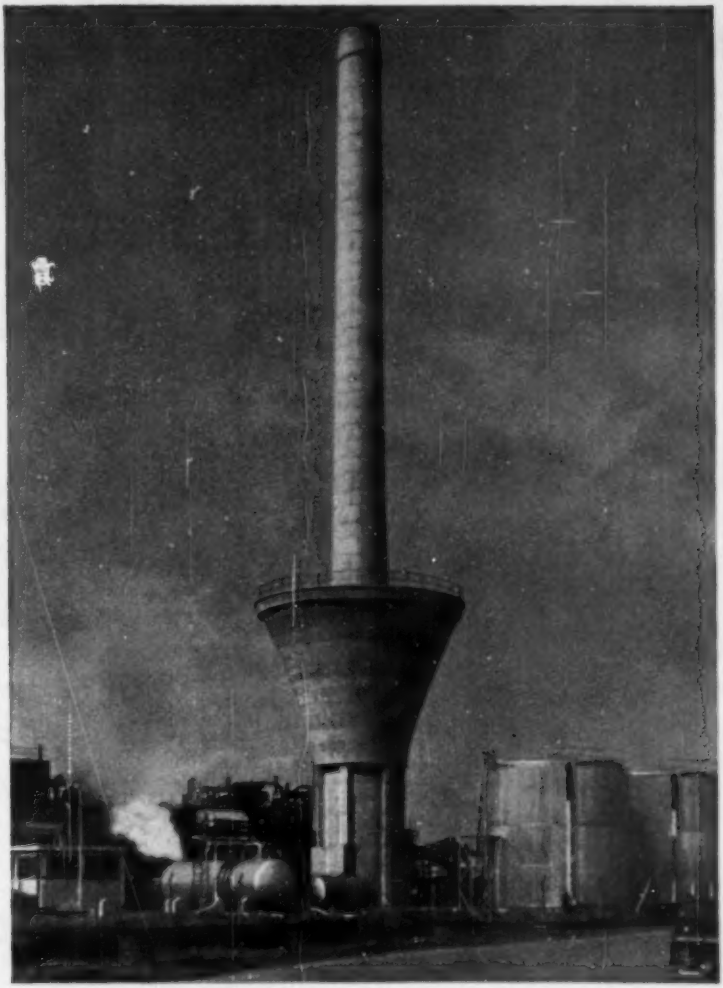

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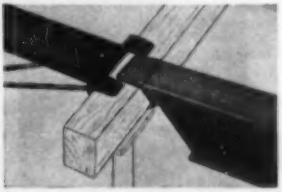

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horizontal shoring which on spans up to 27' 7"

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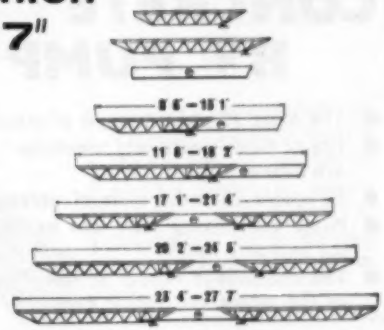
Top bearing sections of the Pecco-Beam have a rolled-in camber. This levels out under load giving a flat surface, but because high grade steel is used to make the beams, the camber remains unaffected even after the shoring has been used hundreds of times.



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Pecco-Beams have two lattice sections, one 8' 6" long, the other 11' 8" long. The plate section is also 8' 6". Spans from 8' 6" to 27' 7" can be obtained by alternating the types of beams used.

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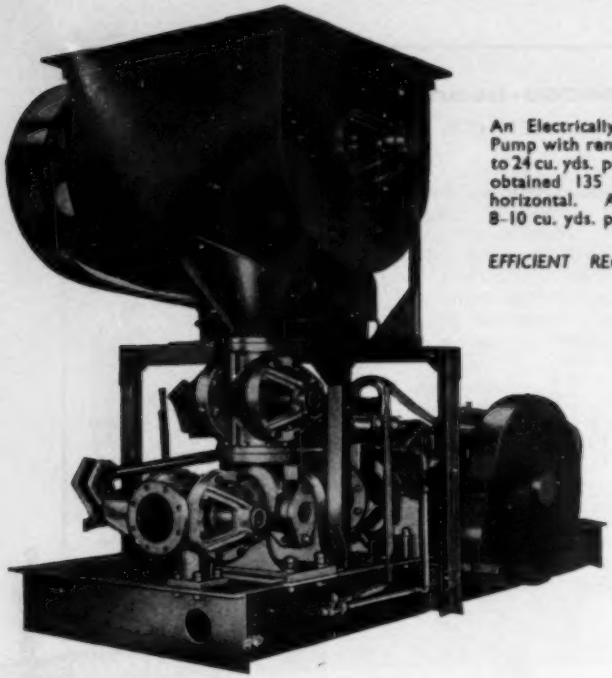

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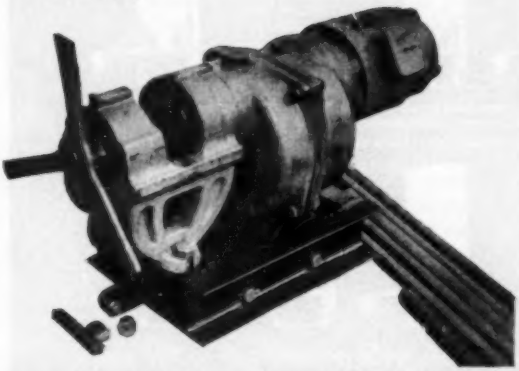

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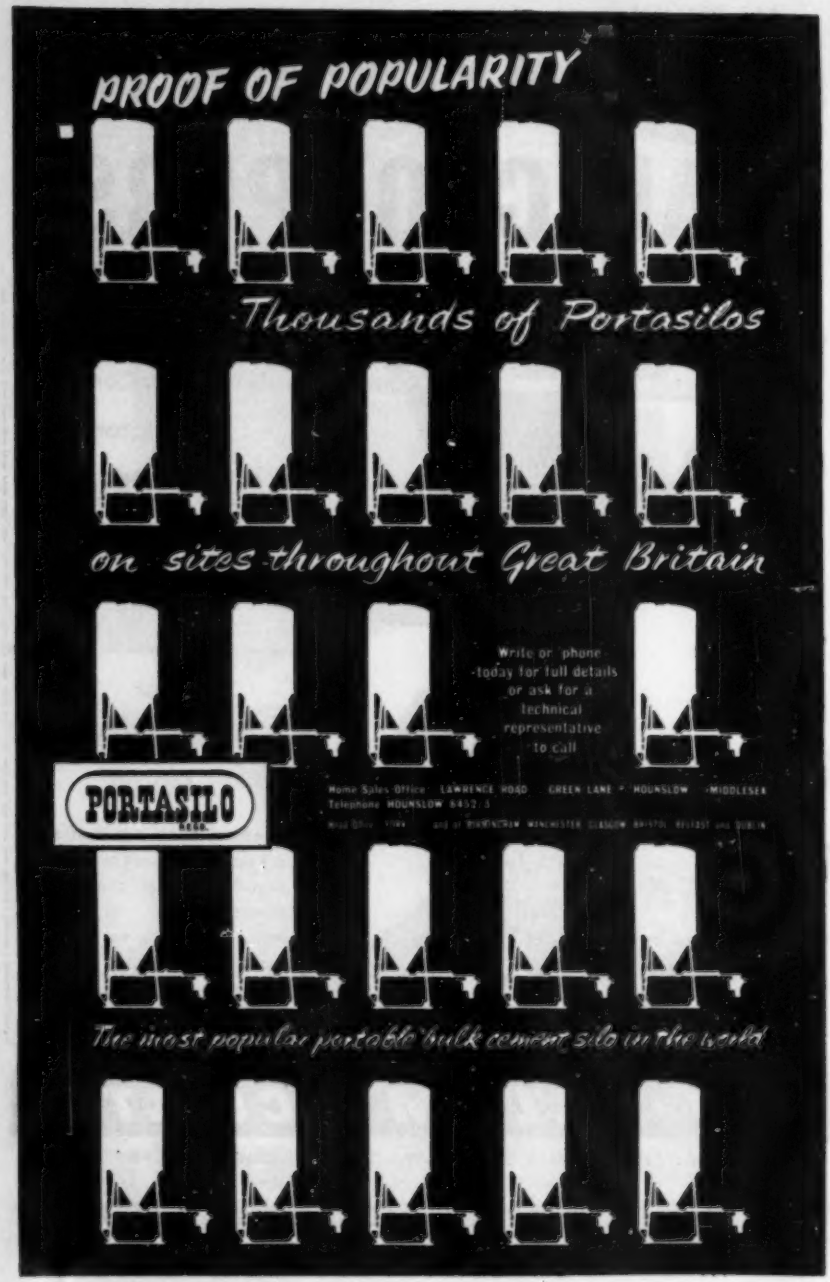
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LVI, No. 4. LONDON, APRIL, 1961.

EDITORIAL NOTES

Engineers: Qualifications and Earnings.

Two recent publications direct attention again to the matters of the training and emoluments of professional engineers. One is a booklet entitled "Professional Engineers" and is issued in the "Choice of Career" series by Her Majesty's Stationery Office. It explains briefly how engineers are trained and gives short descriptions of the work done by different types of engineers including civil, structural and municipal engineers. There were about a hundred thousand professionally qualified engineers in Great Britain in 1959, about 15 per cent. of whom were civil engineers. It is estimated that by 1962 the total needed will represent an increase by a quarter of the present number. Thus it is inferred that a young man entering the profession need have little doubt about the prospects of full employment, and he will be able to, and probably will, change his job several times during his career, mainly to broaden his experience and improve his position.

A young man thinking of taking up engineering must apparently be almost a paragon possessing most of the virtues; according to this booklet, he must be intelligent enough to reason logically, he must have a practical outlook, he must have a mind that is always looking out for the best way of doing things, he must have the judgement to recognise the right solution, and he must be tactful and friendly, yet capable of standing firmly by his opinions. It is encouraging, however, that the authors of the booklet expect an engineer to be able to express himself clearly in speech and writing, for which purpose a good command of English is essential. He is warned that frustrations may be many, and the pressure of work great; an atmosphere of tension is hard to avoid. How many are they that will agree!

Much of the reward for his work will, it is stated, come from his sense of mastery in facing problems, and analysing and solving them; only a small proportion of engineering work calls for the highest academic intellect. A civil engineer looking into a large excavation sees the results of his measurements and calculations as the first stage of his vision of, perhaps, a large power station, and the thought, it is contended, is satisfying. He may even find contentment in the hard work involved in the details of a project. It is well that an engineer

may have these satisfactions because material rewards may not be so generous. The booklet admits that, in Great Britain at least, engineering has not generally been a well-paid profession. (The relative earnings of engineers and other professional men were the subject of the Editorial Note in the number for May 1960.) The number of posts in industrial employment which carry an annual salary exceeding £2000 are few. It is stated that a reasonably successful engineer may expect to receive £800 or more in his mid-twenties and attain double this salary before he is forty; these are no doubt average figures, because in the reinforced concrete industry the general level of salaries seems to be somewhat higher.

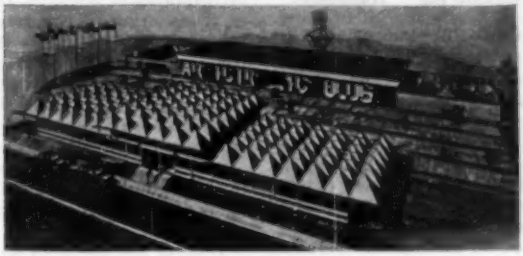
The subject of the remuneration of engineers is dealt with in a report in the second publication, which is the January 1961 number of "The Professional Engineer", the journal of the Engineers' Guild. In particular, the effect of the possession of a university degree on the earnings of an engineer are considered, and the report states that such qualification may enable an engineer at the peak of his career to earn a salary half as much again as that of a non-graduate. The survey, on which the report is based, shows that there has been an improvement of twenty-five to thirty per cent. in salaries since 1956, the percentage increase in the higher incomes being lower. Incomes below £800, a figure stated in the first publication, applied to about eight per cent. of engineers in 1956, whereas in 1960 there was practically none in this class. The number in the higher-income groups, that is over £2400 per annum, had doubled.

The effect of the possession of a university degree is less marked in civil engineering than in other branches, but there is a tendency at all ages and at all levels of salary for a graduate to receive a slightly better remuneration than a non-graduate. Is it that the type of man who voluntarily submits to the discipline of learning attains more naturally the type of mind required of a good engineer? There must, however, be very many with this mental qualification who are not graduates of a university.

The inescapable conclusion is that many employers consider an engineer who can prove his training by attaining academic or institutional qualifications is superior to one without formal training, although there are those of the latter "school" who, from sheer ability, have risen high in their profession. In the main, however, the inference is that the general view seems to accord with that expressed by Sir Isaac Newton in a letter written in 1694 in connection with "A New Scheme of Learning Proposed for the Mathematical Boys in Christ's Hospital":

"A Vulgar Mechanick can practice what he has been taught or seen done, but if he is in error he knows not how to find it out and correct it, and if you put him out of his road, he is at a stand; whereas he that is able to reason nimbly and judiciously about figure, force and motion, is never at rest until he gets over every rub. Experience is necessary, but yet there is the same difference between a mere practical Mechanick and a rational one, as between a mere practical Surveyer or Gager and a good Geometer, or between an Empirick in Physick and a learned and a rational Physitian". [Quoted in "Sir Isaac Newton", by Dr. H. D. Anthony. (1960)]

An Architectural Congress.



THE sixth Congress of the International Union of Architects is to be held in London from July 3 to 7, 1961. The venue of the Congress is to be part of the site of the Dome of Discovery (erected for the Festival of Britain) on the south bank of the River Thames at Lambeth. Two remarkable temporary buildings (see the illustration above) are to be erected. One building will be occupied by an exhibition on the " Architecture of Technology " and two smaller exhibitions, and the other building will be the headquarters of the Congress. The theme of the Congress is to be the manner in which new building materials influence architecture and it is therefore perhaps amazing to find that concrete is not represented in the structural designs of the Congress buildings. The only concrete and cement products to be used extensively are coloured paving slabs for the floors, a form of asbestos-cement sheeting for some of the walls, and glazed concrete blocks for some partitions. (See the note on page 166.)

The main framing of the roof of the headquarters building, which comprises a number of pyramids, is to be supported on hollow steel posts. The roof covering is to be polythene sheeting and large areas of the walls will be of glass. Since one of the principles underlying the construction of the buildings is that they shall be capable of being easily removed after the Congress and re-erected elsewhere, it is surprising that precast structural concrete framing, prestressed perhaps and probably of lightweight concrete, is not intended to be used for at least one, or one part, of the structures. This omission is more surprising since the Congress is devoted to modern building materials and in view of the fact that the Cement and Concrete Association are playing a prominent part in the administrative organisation of the Congress. One cannot help feeling that an opportunity to advance the cause of lightweight precast structural concrete has been missed. Exhibition buildings, because of their temporary nature, can be more experimental and should be more demonstrative of probable developments than more permanent structures. Maybe the exhibitors will, in the materials they display, make up for the omissions of the designers.

It is intended that visitors to the Congress will visit sites and the like to inspect recent developments of structural and decorative concrete. It is hoped that they will be able to see many of the spectacular structures illustrations of which were displayed at an exhibition, " Building in the '60s ", which was organised by the Cement and Concrete Association in London during the last week in March.

Book Reviews.

"Concrete Engineering Handbook." Edited by W. S. Londe and M. F. Janes. (London: McGraw-Hill Publishing Co., Ltd. 1961. £9 14s.)

THIS large "handbook" of upwards of 1200 pages seems to be the successor of the once well-known Hool and Johnson's "Concrete Engineer's Handbook". The treatment of reinforced concrete design is very comprehensive and to a lesser extent so is that of construction. The content, which is the work of twenty contributors, each an authority on his subject, includes materials, cast-insitu and precast construction, design and construction of prestressed concrete, torsion, earthquake resistant structures, bridges of all types, roads, shells, foundations and retaining walls. The long section on building frames and the section on reinforced concrete design (and the numerous related tables) are applicable mainly to practice in U.S.A., but the section on structural theory is of more general application. There is a very good section on chimneys. The treatment of silos includes the theories of Janssen and Airey. The tables and diagrams are very clear and well produced, as is to be expected in books from this particular publisher. After perusal of this book, the question which naturally arises is whether a designer prefers one huge expensive volume covering in a fair degree of detail, as this book does, almost every aspect of concrete engineering, or a number of smaller and less expensive books each of which deals with one aspect only but in perhaps greater detail.

"Versuche über die beim betonieren an den Schalungen entstehenden Belastungen." By O. Graf und F. Kaufmann.

"Druckfestigkeit von Beton in der oberen Zone nach dem Verdichten durch Innenrütler." By K. Walz und H. Schaffler.

"Versuche über die Verdichtung von Beton auf einem Rütteltisch in lose aufgesetzter und in aufgespannter Form." By J. Strey. Deutscher Ausschuss für Stahlbeton. Pamphlet No. 135. (Berlin: Wilhelm Ernst & Sohn. 1960. Price 13 D.M.)

THIS pamphlet of forty-one pages comprises three papers dealing with loads on shuttering, the effects of internal vibration on compressive strength, and the efficiency of compaction of concrete in moulds on vibrating tables. Tests show that when concrete is placed in shuttering of considerable depth, the intensity of pressure on the shuttering is greatest about 5 ft. 6 in. below the surface of the concrete; this effect is attributed to arch action, setting of the lower layers of concrete, and the presence of reinforcement. If the concrete is vibrated the critical depth is greater.

Tests of the effects of internal vibration on the compressive strengths of various parts of prisms of concrete 40 cm. by 40 cm. by 60 cm. high show that the strength of the upper part of the prism is greater than that of the lower part and that compaction for forty-five minutes produced a greater increase of strength of the upper part than of the lower part.

The compaction of concrete in moulds fixed to a vibrating table or resting on the table is the subject of tests described in the third paper. It is shown that greater efficiency is obtained if the mould is loose on the table. An acceleration of $8g$ is required for a fixed mould to produce the same compaction and strength as obtained by an acceleration of $2g$ for a loose mould.

"Kempe's Engineer's Year Book." [London: Morgan Brothers (Publishers) Ltd. 1961. 2 Vols. Price 87s. 6d.]

MOST sections of Kempe's for 1961 have been revised and the section on heating in the "Heating, Ventilation and Air Conditioning" chapter has been rewritten. "Units of Measurement" has been extensively revised, among the additions being a complete range of equivalents and conversion factors for metric and Imperial units. Noteworthy additions have been made to the sections dealing with road construction, foundations, surfacings, and design data for concrete dams. The section on concrete, reinforced concrete, and prestressed concrete is up-to-date in accordance with current British Standards and codes, and in particular, in accordance with Code No. 2007 (1960) for water-containing structures and Code No. 115 (1959) for prestressed concrete in buildings.

Analysis of Frames with V-supports.

By K. STARZEWSKI, Ph.D., B.Sc.

FRAMES with V-supports are used in bridge construction and less frequently for long-span roofs. Unless the slope of the top member of a roof frame is steeper than 1 : 10 it can be neglected. When the inclined columns are slender their flexural stiffness can be neglected. In general, however, the flexural stiffness of the columns should be taken into consideration. An example is given in the following article of the analysis of a symmetrical frame (*Fig.* 1) by the moment-distribution method of analysis; the usual convention of signs is used.

Analysis.

For convenience of analysis the asymmetrical load can be reduced to symmetrical and anti-symmetrical loads as shown in *Fig.* 2. For symmetrical loads there is no sway, but for the anti-symmetrical load, sway has to be taken into consideration and two separate distributions performed. The results are then combined, using the condition that the horizontal shearing force at the level of the beam is zero or equal to the applied force.

Symmetrical Load.—The stiffness K at one end of a prismatic member with the other end fixed is $\frac{4EI}{L}$ and if the other end is hinged is $\frac{3EI}{L}$ in which E is the modulus of elasticity, I is the moment of inertia of the member and L is the distance between the supports. Due to the symmetrical loading shown in *Fig.* 2a, $M_{34} = -M_{43}$. After each distribution, therefore, because of the carry-over effect, bending moment M_{34} is halved. The stiffness of member 3–4 can be modified so that only half the frame need be considered, the carry-over effect still being taken into account. In general, for a member $i - k$ the effective stiffness $K'_{ik} = K_{ik} - K_{ki} \times$ (carry-over factor). In this case $K_{ik} = K_{ki}$ and the carry-over factor is $\frac{1}{2}$, therefore

$$K'_{34} = \tfrac{1}{2}K_{34} = \frac{1}{2} \times \frac{4E \times 5I}{80} = 0{\cdot}125EI.$$

The stiffnesses of other members are as follows:

$$K_{23} = K_{32} = \frac{4E \times 5I}{20} = 1{\cdot}0EI;\ K_{2A} = K_{A2} = K_{3A} = K_{A3} = \frac{4EI}{24{\cdot}2} = 0{\cdot}165EI.$$

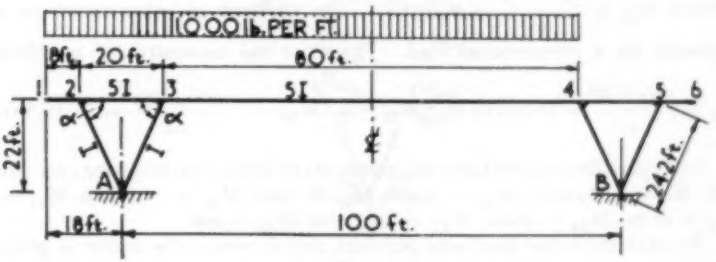


Fig. 1.

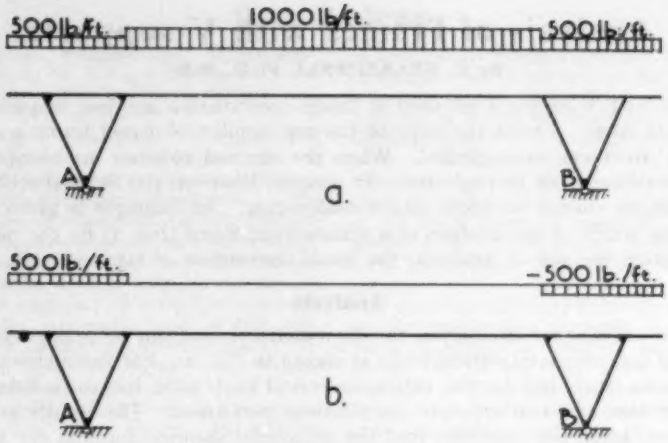


Fig. 2.

The fixed-end moments due to external loading are:

$$M_{21} = \frac{500 \times 8^2}{2} = 16{,}000 \text{ lb.-ft.}; \quad M_{23} = -\frac{500 \times 20^2}{12} = -16{,}700 \text{ lb.-ft.};$$

$$M_{34} = -\frac{1000 \times 80^2}{12} = -533{,}300 \text{ lb.-ft.}$$

The fixed-end moments are distributed in the usual way and the resulting end-moments (in lb.-ft.) are as follows: $M_{21} = +16{,}000$, $M_{2A} = -38{,}000$, $M_{23} = +22{,}000$, $M_{32} = +399{,}000$, $M_{3A} = +74{,}000$, $M_{34} = -473{,}000$, $M_{A2} = -29{,}000$, and $M_{A3} = +28{,}000$.

Anti-symmetrical Load.—Due to the anti-symmetrical load in *Fig.* 2b, $M_{34} = M_{43}$. A plane of contraflexure occurs at the axis of symmetry of the frame and member 3-4 can be treated as having a hinge in the middle. Thus, when evaluating the stiffness of the member, its effective length is half the true length and it can be considered to be hinged at the end, so that the effective stiffness K'_{34} is $\frac{3E \times 5I}{40} = 0{\cdot}375EI$. The stiffness of other members are as evaluated for a symmetrical load. The fixed-end moments are as follows:

$$M_{21} = \frac{500 \times 8^2}{2} = 16{,}000 \text{ lb.-ft.}; \quad M_{23} = -M_{32} = -\frac{500 \times 20^2}{12} = -16{,}700 \text{ lb.-ft.}$$

After distributing the fixed-end moments as before the following end-moments (in ft.-lb.) are obtained: $M_{21} = 16{,}000$, $M_{2A} = 1200$, $M_{23} = -17{,}200$, $M_{32} = 6800$, $M_{3A} = 2000$, $M_{34} = 4800$, $M_{A2} = 800$, and $M_{A3} = 800$.

To determine the fixed-end moment due to sway, the frame is given unit translation at the level of the beam. The respective movements of each joint

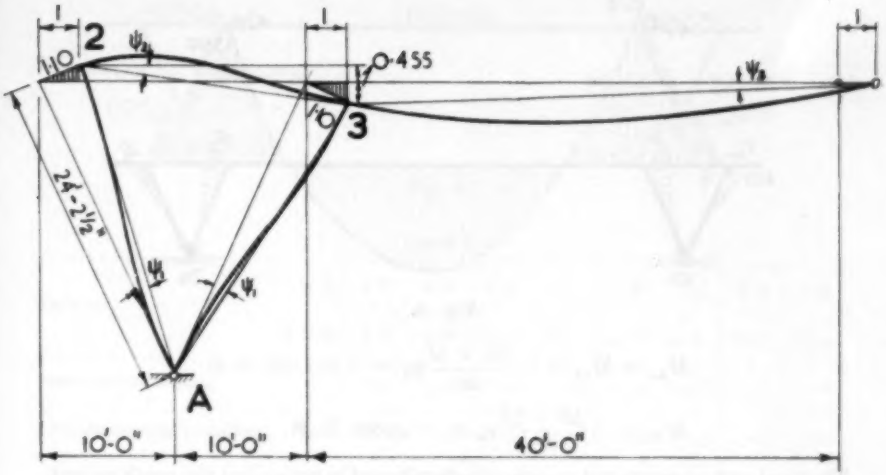


Fig. 3.

due to that translation are shown in *Fig.* 3. From these movements the rotation ψ of each member can be calculated as follows:

$$\psi_1 = \frac{1 \cdot 10}{24 \cdot 2} = 0 \cdot 0455, \; \psi_2 = \frac{0 \cdot 455 + 0 \cdot 455}{20} = 0 \cdot 0455, \text{ and } \psi_3 = -\frac{0 \cdot 455}{40} = -0 \cdot 0114.$$

The fixed-end moments due to unit translation are evaluated from the equations for slope and deflection; for convenience these values are multiplied by $\frac{10^4}{EI}$ and are as follows: $M_{A2} = M_{2A} = M_{A3} = M_{3A} = -\frac{6EI}{24 \cdot 2}\psi_1 = -113{,}000$ lb.-ft.;

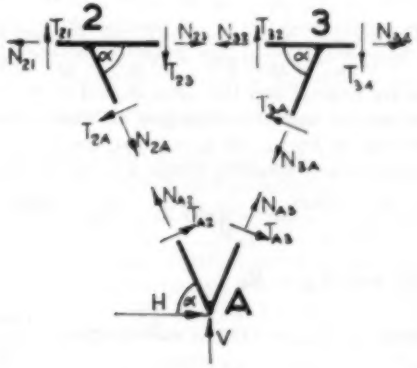


Fig. 4.

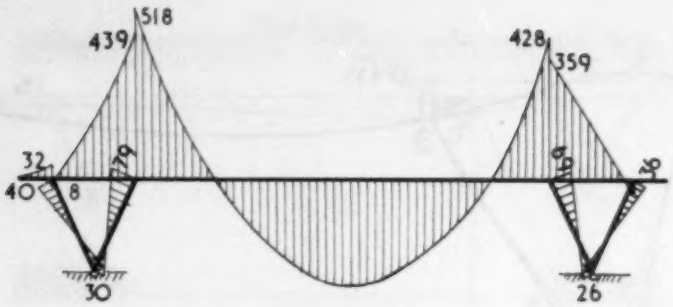


Fig. 5.

$$M_{23} = M_{32} = -\frac{6E \times 5I}{20}\psi_2 = -682{,}000 \text{ lb.-ft.};$$

$$M_{34} = -\frac{3E \times 5I}{80}\psi_3 = +43{,}000 \text{ lb.-ft.}$$

The fixed-end moments due to sway are distributed as before and the resulting end-moments (in lb.-ft.) are: $M_{2A} = 13{,}000$, $M_{23} = -13{,}000$, $M_{32} = -125{,}000$, $M_{3A} = -26{,}000$, $M_{34} = 151{,}000$, $M_{A2} = 10{,}500$, and $M_{A3} = 10{,}500$.

The end-moments produced by the anti-symmetrical load and sway are combined, using the condition that the frame is free to sway and therefore the resulting horizontal shearing force at the level of the beam is zero.

From the end-moments due to the anti-symmetrical load, the horizontal reaction R_1 at the level of the beam, necessary to prevent the frame from swaying, is evaluated by considering the equilibrium of points 2 and 3 as follows: from *Fig.* 4,

$$N_{32} = (T_{34} - T_{32})\cot\alpha - T_{2A}\frac{1}{\sin\alpha} + N_{34};\quad N_{2A} = (T_{32} - T_{34})\frac{1}{\sin\alpha} + T_{3A}\cot\alpha$$

$$N_{23} = T_{2A}\frac{1}{\sin\alpha} + (T_{23} - T_{21})\cot\alpha + N_{21};\quad N_{2A} = (T_{21} - T_{23})\frac{1}{\sin\alpha} - T_{2A}\cot\alpha$$

$$H = (N_{A2} - N_{A3})\cos\alpha - (T_{A2} + T_{A3})\sin\alpha;$$
$$V = (T_{A3} - T_{A2})\cos\alpha - (N_{A2} + N_{A3})\sin\alpha.$$

These equations are general and the signs should be reversed if the forces in a particular case are in the opposite direction to those shown in *Fig.* 4.

For the frame shown in *Fig.* 1, $\sin\alpha = 0{\cdot}909$, $\cos\alpha = 0{\cdot}413$, $\cot\alpha = 0{\cdot}455$. Since joint 3 is in equilibrium, shearing forces T_{ik} can be evaluated as follows:

$$T_{32} = -5000 + \frac{17{,}200 - 6800}{20} = -4480 \text{ lb.},\quad T_{34} = \frac{4800 + 4800}{80} = 120 \text{ lb.},$$

$$T_{3A} = \frac{2850}{24{\cdot}2} = 120 \text{ lb. and } N_{34} = R_1.$$

Therefore, by substitution, $N_{32} = (120 + 4480)0{\cdot}455 - \frac{120}{0{\cdot}909} - R_1$,

or
$$R_1 = N_{32} - 1960 \text{ lb.} \quad . \quad . \quad . \quad . \quad (1)$$

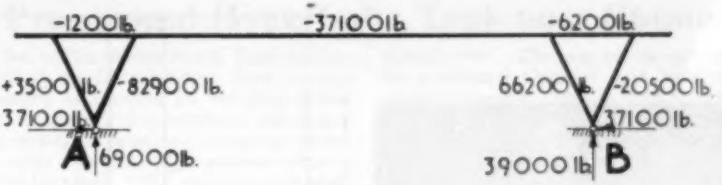


Fig. 6.

TABLE 1.

Member			2-1	2-A	2-3	3-2	3-A	3-4	A-2	A-3
		R.H.S.	5-6	5-B	5-4	4-5	4-B	4-3	B-5	B-4
Symmetrical loading		a	+16	−38	+22	+399	+74	−473	−28	+28
Anti-symmetrical loading		b	+16	−2	−14	+40	+5	−45	−2	+2
Resultant B.M.	a+b	L.H.S.	+32	−40	+8	+439	+79	−518	−30	+30
	−a+b	R.H.S.	0	+36	−36	−359	−69	+428	+26	−26

From the equilibrium of joint 2, $T_{23} = 5000 + \frac{17{,}200 - 6800}{20} = 5520$ lb., $T_{21} = -4000$ lb. and $T_{2A} = -\frac{2000}{24{\cdot}2} = -80$ lb.

Since there is no horizontal force other than R_1 acting on the frame $N_{21} = 0$. Therefore $N_{32} = -\frac{80}{0{\cdot}909} + (5520 + 4000)0{\cdot}455 = 4250$ lb. Substituting in equation (1) $R_1 = 4250 - 1960 = 2290$ lb.

Similarly from the end-moments due to sway and from the equilibrium, at joints 2 and 3, the horizontal reaction R_{11} at the level of the beam, necessary to keep the frame in the swayed condition, can be evaluated and is 8550 lb. Since the total horizontal shearing force at the level of the beam is zero, $R_1 + \gamma R_{11} = 0$ or $\gamma = \frac{R_1}{R_{11}} = -0{\cdot}268$. The bending moments due to unit translation of the frame are therefore multiplied by −0·268 and added to those due to anti-symmetrical external loading to give the bending moments in lb.-ft. due to the anti-symmetrical loading as follows: $M_{21} = 16{,}000$, $M_{2A} = -2300$, $M_{23} = -13{,}700$, $M_{32} = +40{,}300$, $M_{3A} = +5000$, $M_{34} = -45{,}300$, $M_{A2} = -1900$, $M_{A3} = +1900$. The results of the distribution of symmetrical and anti-symmetrical bending moments are combined to give the final bending moments for the frame as shown in *Table* 1 and *Fig.* 5. The forces in all the members and the horizontal and vertical reactions are evaluated, by considering the equilibrium of the joints, and are shown in *Fig.* 6 in which a negative sign indicates compression.

The bending moments in *Table* 1 are in units of 1000 lb.-ft.

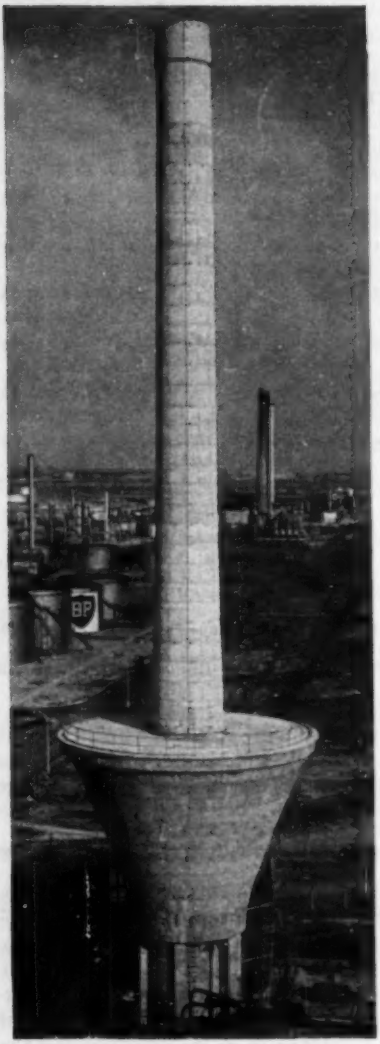


Fig. 1.

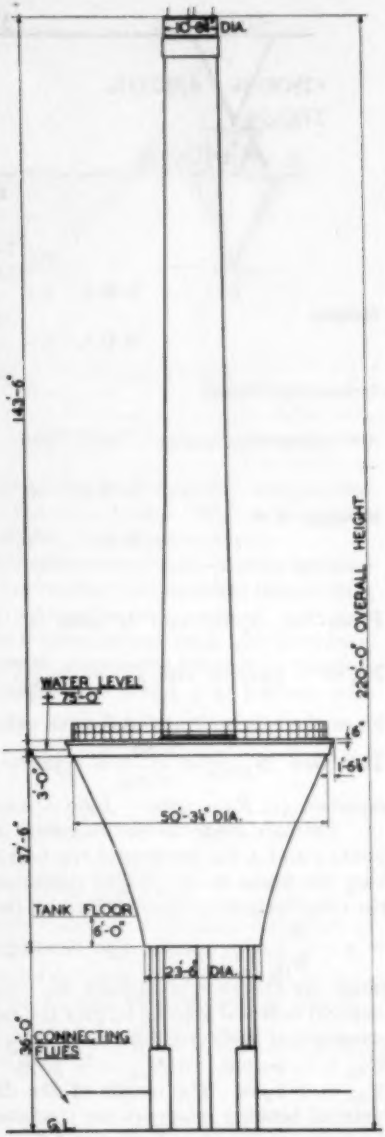


Fig. 2.

See facing page.

A Prestressed Hyperbolic Tank on a Chimney.

At the works of the South Eastern Gas Board at the Isle of Grain, Kent, it was necessary to provide an elevated water tank to contain demineralised water and to be as near as possible to the place where this water was to be used, namely, close to the boiler plant. The requirements were that the tank should contain 150,000 gallons, and, to suit the capacity of the pumps of the water-treatment plant, the level of the water in the tank had not to exceed 75 ft. and the level of the delivery outlet had to be not more than 42 ft. above the ground. There are also restrictions on the site because of other plant which prevented an ordinary water tower being constructed adjacent to the boiler plant. Since a new chimney was required in connection with the boiler plant, a solution to the problem was to support the tank on the shaft of the chimney. This design increases slightly the volume of concrete in the base and shaft of the chimney because the provision of pilasters is desirable for strengthening the shaft, especially as large openings, diametrically opposed, are required in the shaft for ingress of the flues.

Ordinary prestressed concrete tanks contain vertical and circumferential prestressing cables or wires. To achieve economies in an elevated structure it was considered that inclined cables, in the tank wall and anchored at the top and bottom and stressed from the top, would considerably reduce the cost of circumferential prestressing in spite of the probable increase in the number of cables. It was considered that an annular tank with a flat bottom was undesirable, not only aesthetically but structurally, on account of the tensile forces which might be developed in the bottom. The structure (*Figs.* 1 and 2) as finally designed and erected comprises a reinforced concrete chimney, 220 ft. high, and a hyperbolic annular prestressed concrete tank, 40 ft. 6 in. deep. The chimney shaft has a diameter of 19 ft. at the base and 10 ft. 8¾ in. at the top. The external diameter of the tank is 23 ft. 6 in. at the base and 50 ft. 3¼ in. at the top. The structure is supported on sixty-two cast-insitu bulb-based piles.

By combining the chimney and tank there is a saving of about twenty piles and 85 cu. yd. of concrete which would be required for the substructure of a separate water tower. The size of the pile-cap of the combined chimney and tank is increased, however, from a cylindrical slab 29 ft. in diameter and 5 ft. thick to an octagonal base 33 ft. across the flats and 6 ft. thick. The net saving is therefore in the number of piles and the shuttering of the substructure, the cost of the extra concrete in the base being about equal to that which would be required in the substructure of a separate water tower.

The tank, details of which are given in *Figs.* 3, 4 and 5, is supported freely on a peripheral corbel, which is carried on six pilasters 11½ in. wide and projecting from and monolithic with the shaft. A sheet of lead is provided as a seating for the tank (*Fig.* 5). The base of the tank is 6 ft. thick; it is not of this thickness for structural reasons, but to provide a reasonable minimum width to the annular space at the bottom of the tank, and incidentally to accommodate the anchor-loops of the prestressing cables. The inner wall of the tank is 6 in. thick and is separated from the shaft of the chimney by a ¾-in. gap, which is filled by a ½-in. sheet of heat-resisting joint-filler laid against the chimney shaft and a ¼ in. of ordinary flexible joint-filler against the tank wall. The outer wall of the tank is 12 in. thick above the base but reduces to a general thickness of 8½ in. up to the top where a capping-beam 2 ft. 6 in. deep and about 14 in. wide is provided.

The roof of the tank is a reinforced concrete slab having an average thickness of 7 in. and supported on the capping-beam and the inner wall. A sheet of lead is provided between the roof and its supports. Access to the top of the tank is provided by means of a D-shaped shaft extending vertically through the tank and cast monolithically with the inner wall. A covered manhole in the roof is provided for entry to the tank. Guard-rails and toe-boards are provided round the periphery of the roof.

The outer wall of the tank is prestressed by means of two identical series of inclined cables, there being sixty post-tensioned cables in each series. Each cable comprises eight 0·276-in. wires and is contained in a flexible metal sheath except, as shown in *Fig.* 5, where the sheaths are cut to enable the wire to be spread out and the cables looped and embedded in

the bottom slab. The loops (*Fig.* 4) are 4 ft. 6 in. long and 1 ft. 6 in. wide. The inner series of cables are wound clockwise and the outer series anti-clockwise. The inclination of the cables is such as to form at any point an angle of 38 deg. with the tangent to the hyperbola. The cables are stressed from, and anchored in, the capping-beam; therefore a rebate 3½ in. wide and 5 in. deep was left in the capping-beam and was filled in with concrete after the cables had been tensioned and anchored. Helical reinforcement is provided around the anchorages of the cables (but is not shown in *Fig.* 5). Bleed tubes are provided at the lower end of the sheaths so that the ducts can be grouted after the construction of the wall.

The inner wall and access shaft are prestressed with thirty-three vertical post-tensioned cables; each cable comprises four 0·276-in. wires and is contained in a square sheath. All the cables, except one, are continuous in pairs to provide a semicircular loop for anchoring in the bottom of the tank.

Each wire in the cables was stressed with a force of not less than 8200 lb., which should give an extension, after losses, of 2 $\frac{7}{16}$ in. for each wire of the cables in the outer wall and 1 $\frac{11}{16}$ in. for each wire of the cables in the inner wall. A complex sequence of stressing was required to ensure that practically a uniform stress was induced. Jacking was carried out simultaneously from four points and, after experience had been gained, one complete cable was stressed in about 30 minutes.

The chimney shaft is lined throughout with acid-resisting bricks set in acid-resisting cement paste. The lining is supported on corbels monolithic with the shaft at 20-ft. and 24-ft. centres. An air space of 4-in. minimum width is provided between the shaft and the lining, and is ventilated to the atmosphere by a series of openings, 4 in. in diameter and arranged

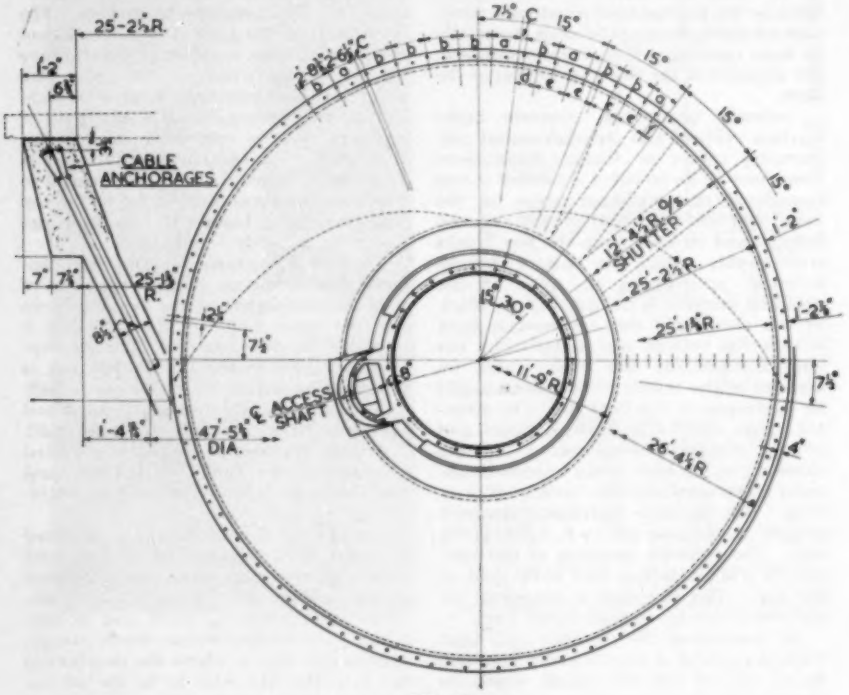


Fig. 3.

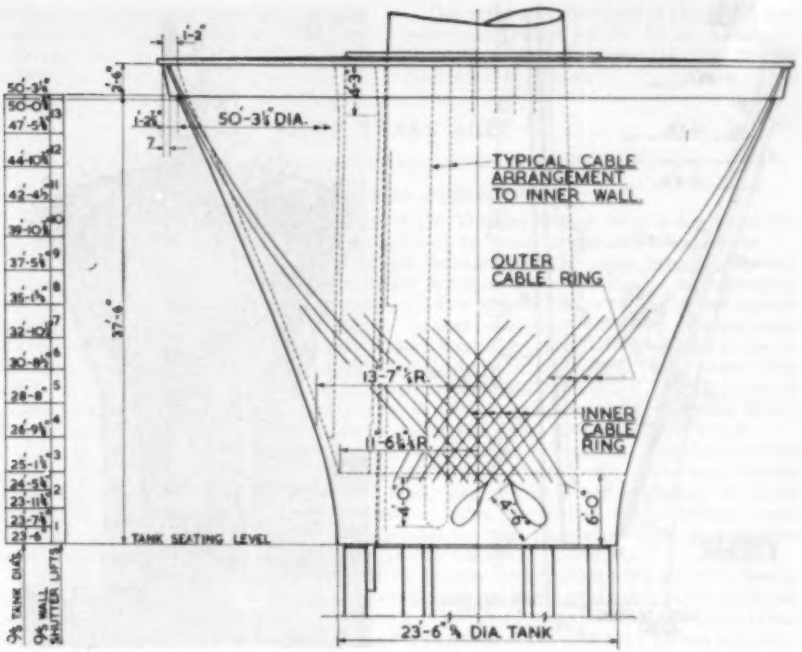


Fig. 4.

circumferentially, through the shaft just below the corbels. Since vents could not be provided where the tank abuts the shaft, ventilation is provided by vertical holes 4 in. in diameter formed in the corbels and connecting the series of air spaces. Circulation in the space at the tank is therefore from the external vents immediately below the tank to those immediately above. This ensures that the temperature of the shaft is not excessive and that the minimum amount of heat is transferred to the tank.

To prevent contamination of the demineralised water by the silica compounds in the cement, the internal surfaces of the tank are coated with a thick layer of bitumen solution, which was applied before the tank was tested for watertightness. On filling the tank for the first time a few patches of damp were evident on the external wall and a slight leak in the inner wall was detected. It was apparent that the leaks were occurring at the construction joints in the lower half of the tank. Remedial measures were not easy because of the internal bitumen coating, but finally the leaks were cured by cleaning off this material at the construction joints and applying a band of glass-fibre polyester laminate to the surface of the concrete. This treatment was fully effective on the internal wall but inadvertently the bands were not continued around the access shaft where patches of damp remained until treated externally with a water-sealing compound.

The concrete used for the chimney and tank was mixed in the proportions of 1 : 1·91 : 3·5 with a water-cement ratio of 0·505, ordinary Portland cement being used throughout.

The walls of the tank were cast in thirteen lifts, each about 3 ft. deep as shown in *Fig.* 4. An external metal-tube scaffold was erected around the structure as shown in *Fig.* 6. The panels of wooden shuttering are shown in *Fig.* 7, in which also is seen the access platform leading from a hoist to the working positions.

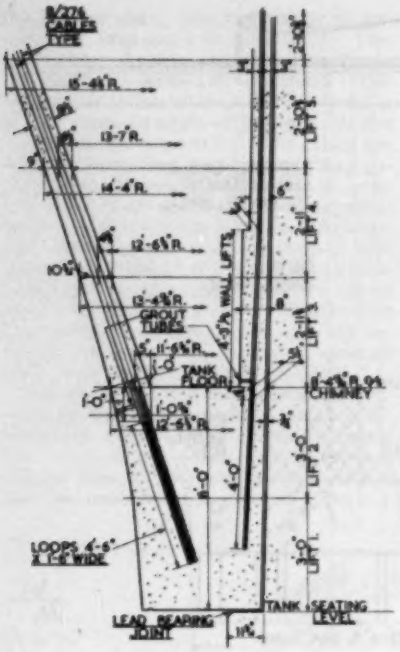


Fig. 5.

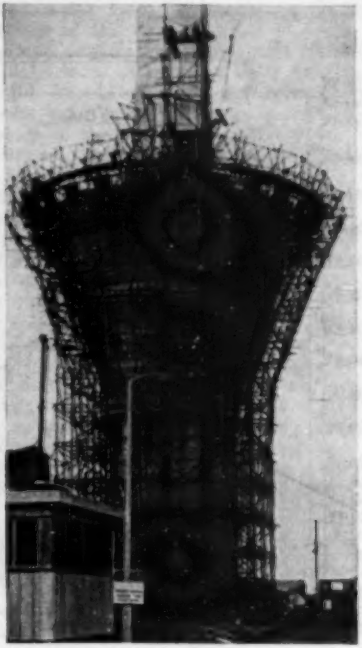


Fig. 6.

The total cost of the chimney and tank, including the piled foundations, was £40,000. The tank was designed by the South Eastern Gas Board in collaboration with Messrs. Tileman & Co., Ltd., who also designed the chimney and were the contractors for the chimney and tank. The foundation work was constructed by

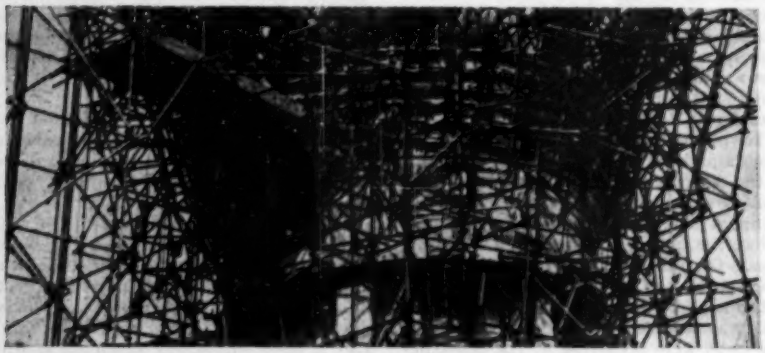


Fig. 7.

Sir Robert McAlpine & Sons Ltd., on piles provided by Frankipile Ltd. The prestressing was by the P.S.C. Mono-wire system.

The notes and drawings in the foregoing were contributed by Mr. D. A. Andrews, M.I.C.E., Civil Engineer of the South Eastern Gas Board.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", April, 1911.

Coke Breeze as Aggregate.

The following extracts give the views of Mr. Thomas Potter, who is described (in 1911) as the "doyen of the concrete industry", on coke breeze as an aggregate.

"There is one aggregate many thought beneath notice—coke breeze. It was extensively used for many years, principally for floors, but, I submit, undeservedly lost its character some time since through a letter in the *Times* telling of its failure in Germany, where it had been prohibited. It was not stated, I believe, of what class of coal it formed the residue, and this is an important point. I have seen some in Scotland totally unfit for concrete purposes, but not in England. This letter was followed by others telling how it was of such an inflammable nature, that if exposed to a severe fire it was second only to coal in assisting combustion. Another letter stated that it disintegrated in the course of a few years and crumbled away.

As to inflammability, the tests made by the Fire Prevention Committee with various kinds of floors, each under similar conditions exposed for three hours to an increasing and ultimate temperature of 1,900 deg. F., and water played on them from a hose pipe, gave the following results. Coke breeze and burnt ballast equal and best; each was found, after being allowed to cool, practically free from cracks or deflection, and Thames ballast and granite chips the worst.

As to disintegration, this would not be readily ascertained perhaps with floors, which are usually plastered on the underside and covered with some other material on the surface, but much of the facing and all the ornamental work of the large block of buildings numbers 64 to 70 Victoria Street, London, are of cast coke breeze concrete, executed about forty years ago. No facing was done to this work after it left the moulds; the coke breeze is within an eighth of an inch of the surface, and I believe there has never been a crack, fracture, or any sign of disintegration at any time.

I have used some thousands of tons of coke breeze for floors, and do not remember a failure of any kind therewith. I am aware that coke breeze is condemned by many as an unsuitable aggregate. I can only speak from my own experience."

[Mr. Thomas Potter was one of the better known writers on concrete in the late nineteenth century, his best-known book, which was published in 1877 when he was clerk of works to Lord Ashburton, being "Concrete: Its Use in Building and the Construction of Concrete Walls, Floors, etc." It contains much sound advice on various matters, such as the quantity of water.

As is well known, coke breeze is a gas-works product containing much combustible material and is used as a fuel by blacksmiths and others, and is today unreservedly condemned as an aggregate for concrete. It is possible that Mr. Potter was confusing inferior coke breeze with superior clinker, a furnace residue (not ash) which, so long as the content of combustible material does not exceed that specified in B.S. 1165 (1957), is suitable for lightweight non-reinforced concrete.

The buildings in Victoria Street, London, the façades of which are mentioned in the foregoing extracts, still stand, but they are due for early demolition; although over ninety years old the facings are at present in tolerably good condition.

The British Fire Prevention Committee, which was, we are informed, founded about 1909, was reconstituted after the World War II and some of its functions were taken over by the Fire Offices' Committee, while the testing work was taken over by the Fire Research Organisation established jointly by the Department of Scientific and Industrial Research and Fire Offices' Committee. In 1946, the Fire Offices' Committee established the Fire Offices Committee Protection Association which, from February 20, 1961, adopted the shortened title of the Fire Protection Association.]

The Highest Residential Building in London.

THE construction of the tallest block of flats to be built in London to date is now nearing completion (*Fig.* 1). The building, which is of twenty-three storeys, is 212 ft. in height and is at Hide Place, Westminster. The structure is of reinforced concrete which up to the second-floor level is of cast-insitu concrete and above this of precast concrete frames. The frames are anchored to the central core of the building, which houses the services, by precast concrete floor slabs. Each floor slab weighs 3 tons and is 138 ft. square; there are thirty such slabs in each floor. The shortest time of construction of a floor was four days. The heating apparatus is to be installed under the floors and will heat the concrete by a system of wires. The wall panels are precast (*Fig.* 2), the external finish being exposed Norwegian quartz aggregate.

Fig. 1.

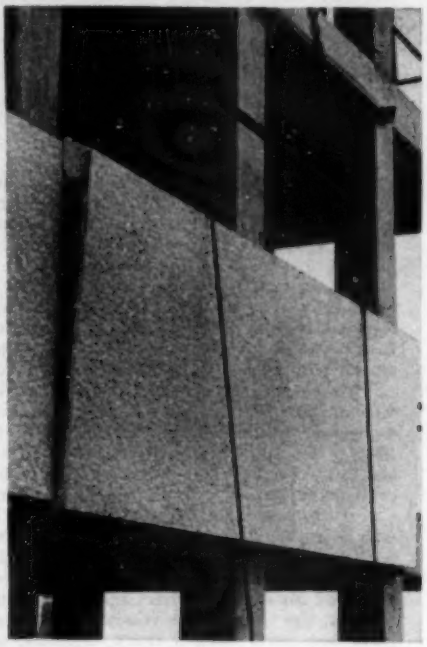


Fig. 2.

A feature of the constructional plant is a travelling tower-crane which is the highest in Britain, the top being 213 ft. above the ground; the crane cannot be used should the wind velocity exceed 18 miles per hour.

The building is being erected for the Westminster City Council. The architects are Messrs. Stillman and Eastwick-Field; the structural engineers are Messrs. Charles Weiss and Partners. The contractors are Messrs. John Laing & Son, Ltd.

Drawing Office Exhibition.

A DRAWING Office Equipment and Materials Exhibition is to be held in the Royal Horticultural Society's New Hall, Westminster, from June 5 to 8, 1961.

Moment-distribution Factors for Members with Change of Cross-section.

By A. ORMEROD, B.Sc., D.I.C., A.M.I.Mech.E.

IN the analysis of a structure by the method of moment distribution, carry-over factors, distribution factors and fixed-end moments of component members having one or more abrupt changes of cross-section can be determined readily by the following method and formulæ.[1]

Basic Formulæ.

Cantilever with Moment Applied at the Free End.—One expression for $\frac{M}{EI}$ valid for the entire length of the beam shown in *Fig.* 1a may be written by incorporating an operator k which may be either positive or negative and which has the following values; if $x < a$, $k = 0$; if $x > a$, $k = \frac{I_1}{I_2} - 1$, in which x is the distance from the free end of the cantilever to the section under consideration. Then at any distance x, $\frac{M}{EI} = \frac{M_0}{EI_1}(1 + k)$ and, by integration,

$$EI_1\frac{dy}{dx} = M_0x(1 + k) - M_0ak + A.$$

$$EI_1y = M_0\frac{x^2}{2}(1 + k) - M_0akx + M_0\frac{a^2}{2}k + Ax + B.$$

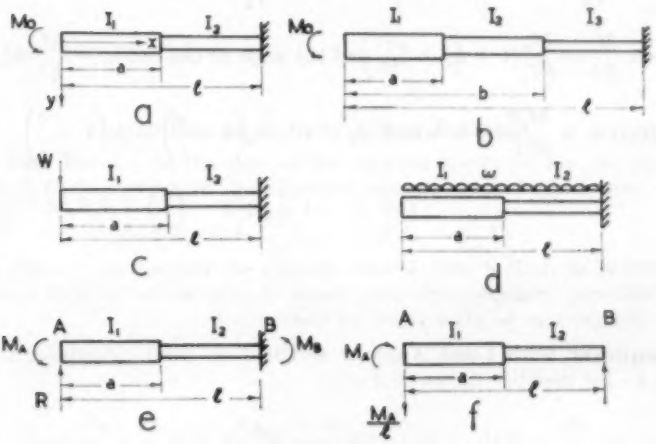


Fig. 1.

[1] Reference should also be made to "The Treatment of Discontinuities in Beam Deflection Problems". By C. L. Brown. Quarterly Journal of Applied Mathematics. Vol. 1. 1944. P. 349.

At each integration terms containing k are added to ensure that when x equals a there is no sudden change in the value of slope or deflection given by the formulæ. When $x = l$ slope and deflection are zero and the following values of A and B are obtained,

$$A = -M_0 l\left[1 + k\left(1 - \frac{a}{l}\right)\right];\ B = M_0\frac{l^2}{2}\left\{1 + k\left[1 - \left(\frac{a}{l}\right)^2\right]\right\}.$$

It is convenient to express results in terms of a series of factors which for a member with only one change of cross-section are as follows.

$$k = \frac{I_1}{I_2} - 1;\ k_1 = 1 + k\left(1 - \frac{a}{l}\right);$$

$$k_2 = 1 + k\left[1 - \left(\frac{a}{l}\right)^2\right];\ k_3 = 1 + k\left[1 - \left(\frac{a}{l}\right)^3\right],\ \text{etc.}$$

Thus the slope at the free end is $-\frac{M_0 l}{EI_1}k_1$, ... (1)

and the deflection at the free end is $\frac{M_0 l^2}{2EI_1}k_2$. . . . . (2)

If there is more than one change of cross-section (*Fig.* 1b) it is necessary to introduce factors for each change. Suitable factors are

$$k_a = \frac{I_1}{I_3} - 1 \text{ (when } x > a\text{), and } k_b = \frac{I_1}{I_3} - 1 - k_a \text{ (when } x > b\text{).}$$

Then $\frac{M}{EI} = \frac{M_0}{EI_1}(1 + k_a + k_b)$, and the slope at the end is $-\frac{M_0 l}{EI_1}k_1$, and the deflection is $\frac{M_0 l^2}{2EI}k_2$, in which $k_1 = 1 + k_a\left(1 - \frac{a}{l}\right) + k_b\left(1 - \frac{b}{l}\right)$,

and $k_2 = 1 + k_a\left[1 - \left(\frac{a}{l}\right)^2\right] + k_b\left[1 - \left(\frac{b}{l}\right)^2\right]$.

This method of dealing with several changes of cross-section is quite general. In the following examples only one change of cross-section is dealt with. Any further changes can be allowed for by summation.

Cantilever with Load Applied at the Free End.—Applying the same process to the member shown in *Fig.* 1c,

$$EI_1\frac{d^2y}{dx^2} = Wx(1 + k), \quad EI_1\frac{dy}{dx} = W\frac{x^2}{2}(1 + k) - W\frac{a^2}{2}k + A,$$

$$\text{and } EI_1 y = W\frac{x^3}{6}(1 + k) - W\frac{a^2}{2}kx + W\frac{a^3}{3}k + Ax + B.$$

At the free end the slope is $-\frac{Wl^2}{2EI_1}k_2$, ... (3)

and the deflection is $\frac{Wl^3}{3EI_1}k_3$. . . . . (4)

Cantilever with a Uniformly-distributed Load.—Referring to *Fig.* 1d,

$$EI_1\frac{d^2y}{dx^2} = \frac{wx^2}{2}(1+k), \quad EI_1\frac{dy}{dx} = \frac{wx^3}{6}(1+k) - \frac{wa^3}{6}k + A,$$

$$\text{and } EI_1y = \frac{wx^4}{24}(1+k) - \frac{wa^3}{6}xk + \frac{wa^4}{8}k + Ax + B.$$

At the free end, the slope is $-\frac{wl^3}{6EI_1}k_3$, ... (5)

and the deflection is $\frac{wl^4}{8EI_1}k_4$. . . . . (6)

Determination of Carry-over Factors.

Considering the member shown in *Fig.* 1e, and which is hinged at A, R is evaluated by combining equations (2) and (4) to give zero deflection at A.

Thus, $\frac{1}{2}M_Al^2k_2 = \frac{1}{3}Rl^3k_3$ or $R = \frac{3}{2}\frac{M_A}{l}\frac{k_2}{k_3}$,

and $M_B = Rl - M_A = M_A\frac{1}{2}\left(\frac{3k_2}{k_3} - 2\right)$.

Thus the carry-over factor from A to B is $\frac{1}{2}\left(\frac{3k_2}{k_3} - 2\right)$ . . . . (7)

Determination of Distribution Factors.

One End Fixed.—In the case of the member shown in *Fig.* 1e, the net slope θ at A is determined by combining equations (1) and (3) to give

$$EI_1\theta = M_Alk_1 - \frac{Rl^2}{2}k_2 = M_Al\left(k_1 - \frac{3}{4}\frac{k_2^2}{k_3}\right)$$

or $\frac{M_A}{E\theta} = \frac{4I_1}{l}\frac{k_3}{4k_1k_3 - 3k_2^2}$. . . . . (8)

Hinged Ends.—Referring to *Fig.* 1f,

$$EI_1\frac{d^2y}{dx^2} = M_A\left(\frac{x}{l} - 1\right)(1+k).$$

$$EI_1\frac{dy}{dx} = M_A\left(\frac{x^2}{2l} - x\right)(1+k) - M_A\frac{a^2}{2l}k + M_Aak + A.$$

$$EI_1y = M_A\left(\frac{x^3}{6l} - \frac{x^2}{2}\right)(1+k) - M_A\frac{a^2}{2l}kx + M_Aakx + M_A\frac{a^2}{3l}k - M_A\frac{a^2}{2}k + Ax + B.$$

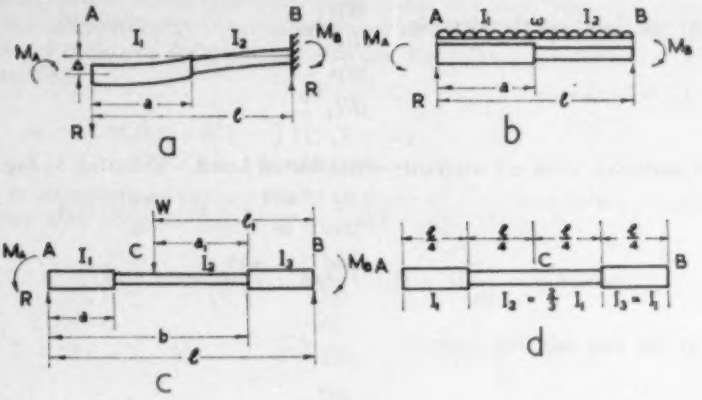


Fig. 2.

When $x = 0$, $y = 0$, therefore, $B = \frac{M_A a^2 k}{6l}(3l - 2a)$. Since x is less than a, $k = 0$; therefore $B = 0$.

When $x = l$, $y = 0$, therefore $A = \frac{M_A l}{3}\left[3k_1 - 3k_2 + k_3\right]$;

therefore
$$\frac{M_A}{E\theta} = \frac{3I_1}{3l}\left[\frac{1}{3(k_1 - k_2) + k_3}\right]. \quad . \quad . \quad . \quad . \quad (9)$$

Sidesway.

Both Ends Fixed.—In the case of the member shown in *Fig.* 2a, R can be evaluated by combining equations (1) and (3), since the slope at A is zero,

$$M_A l k_1 = R\frac{l^2}{2}k_2 \quad \text{or} \quad R = 2\frac{M_A}{l}\frac{k_1}{k_2},$$

and considering the deflection at A, from equations (2) and (4)

$$EI_1\Delta = \frac{Rl^3}{3}k_3 - \frac{M_A l^2}{2}k_2 = \frac{M_A l^2}{6}\frac{4k_1k_3 - 3k_2^2}{k_2}$$

or
$$M_A = 6E\Delta\frac{I_1}{l^2}\frac{k_2}{4k_1k_3 - 3k_2^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

and
$$M_B = Rl - M_A = 6E\Delta\frac{I_1}{l^2}\frac{2k_1 - k_2}{4k_1k_3 - 3k_2^2}. \quad . \quad . \quad . \quad (11)$$

If the end A of the member in *Fig.* 2a is hinged, equation (4) gives,

$$M_B = 3E\Delta\frac{I_1}{l^2}\frac{1}{k_3} \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Determination of Fixed-end Moments.

Uniformly-distributed Load.—For the member shown in *Fig.* 2b, equations (1) to (6) are combined for the case in which the slope and deflection at A are zero. This leads to the following equations.

$$R\frac{l^2}{2}k_2 - M_A l k_1 - \frac{wl^3}{6}k_3 = 0$$

$$R\frac{l^3}{3}k_3 - M_A\frac{l^2}{2}k_2 - \frac{wl^4}{8}k_4 = 0.$$

Solving these equations, $$M_A = \frac{wl^2}{12}\frac{9k_2k_4 - 8k_3^2}{4k_1k_3 - 3k_2^2} \quad . \quad . \quad . \quad . \quad (13)$$

and $$R = \frac{wl}{2}\frac{3k_1k_4 - 2k_2k_3}{4k_1k_3 - 3k_2^2},$$

Since $$M_B = M_A - Rl + \frac{wl^2}{2},$$

$$M_B = \frac{wl^2}{12}\left(\frac{4k_3(3k_2 - 2k_3) - 9k_4(2k_1 - k_2)}{4k_1k_3 - 3k_2^2} + 6\right). \quad . \quad . \quad (14)$$

Concentrated Loads.—Referring to *Fig.* 2c, a concentrated load is regarded as a load on the end of a cantilever CB. The slope and deflection at C are determined as a means of finding the slope and deflection at A. It is necessary to calculate the factors for the changes of cross-section in CB. The basic formulæ will give the slope and deflection at C in terms of EI_2. It is convenient to have these quantities in terms of EI_1 and the conversion is best made to the factors themselves. Thus for the cantilever CB,

$$k = \frac{I_2}{I_3} - 1, \quad k_1 = 1 + k\left(1 - \frac{a_1}{l_1}\right), \text{ etc.}, \quad k' = \frac{I_1}{I_2}k, \quad \text{and} \quad k'_1 = \frac{I_1}{I_3}k_1, \text{ etc.}$$

When the factors have been calculated the net slope and net deflection at A are equated to zero, giving $$\frac{Rl^2}{2}k_2 - M_A l k_1 - \frac{Wl_1^2}{2}k'_2 = 0$$

and $$\frac{Rl^3}{3}k_3 - M_A\frac{l^2}{2}k_2 - \frac{Wl_1^3}{3}k'_3 - W\frac{l_1^2}{2}k'_2(l - l_1) = 0.$$

Solving these equations,

$$M_A = W\frac{l_1^2}{l^3}\frac{lk'_2(3k_2 - 2k_3) - l_1k_2(3k'_2 - 2k'_3)}{4k_1k_3 - 3k_2^2} \quad . \quad . \quad (15)$$

and $$R = \frac{Wl_1^2}{l^3}\frac{3lk'_2(2k_1 - k_2) - l_1 2k_1(3k'_2 - 2k'_3)}{4k_1k_3 - 3k_2^2},$$

Since $M_B = M_A - Rl + Wl_1$

$$M_B = W\frac{l_1}{l^2}\left[\frac{l_1^2(2k_1 - k_2)(3k'_2 - 2k'_3) - 2ll_1k'_2(3k_1 - 3k_2 + k_3)}{4k_1k_3 - 3k_2^2} + l^2\right]. \quad . \quad (16)$$

Example.

Determine the distribution factor, carry-over factor and the fixed-end moments for the beam in *Fig.* 2d, subjected to a uniformly-distributed load wl or a concentrated load W.

$$k_a = +\frac{1}{2}, \qquad k_b = -\frac{1}{2}, \qquad k_1 = 1 + \frac{1}{2}\left(1 - \frac{1}{4}\right) - \frac{1}{2}\left(1 - \frac{3}{4}\right) = \frac{5}{4},$$

$$k_2 = 1 + \frac{1}{2}\left(1 - \frac{1}{16}\right) - \frac{1}{2}\left(1 - \frac{9}{16}\right) = \frac{5}{4}, \qquad k_3 = 1 + \frac{1}{2}\left(1 - \frac{1}{64}\right) - \frac{1}{2}\left(1 - \frac{27}{64}\right) = \frac{77}{64},$$

$$k_4 = 1 + \frac{1}{2}\left(1 - \frac{1}{256}\right) - \frac{1}{2}\left(1 - \frac{81}{256}\right) = \frac{37}{32}.$$

The distribution factor for fixed ends is determined from formula (8).

$$\frac{M_A}{E\theta} = \frac{4I_1}{l} \times \frac{77}{64} \div \left[\left(5 \times \frac{77}{64}\right) - \frac{75}{16}\right] = \frac{I_1}{l} \times \frac{308}{85}.$$

The carry-over factor for fixed ends is determined from formula (7) and is

$$\frac{1}{2}\left[\left(\frac{15}{4} \div \frac{77}{64}\right) - 2\right] = \frac{43}{77}.$$

For a load wl uniformly distributed over the span the fixed-end moment is calculated from formula (13).

$$M_A = \frac{wl^2}{12} \frac{\left(9 \times \frac{5}{4} \times \frac{37}{32}\right) - \left(8 \times \frac{77}{64} \times \frac{77}{64}\right)}{\left(4 \times \frac{5}{4} \times \frac{77}{64}\right) - \left(3 \times \frac{25}{16}\right)} = \frac{wl^2}{12} \times \frac{43}{40}.$$

If the beam carries a central point load W, for the cantilever CB

$$k = -\frac{1}{3}; \quad k' = \frac{3}{2}\left(-\frac{1}{3}\right) = -\frac{1}{2}; \quad k'_1 = \frac{3}{2}\left[1 - \frac{1}{3}\left(1 - \frac{1}{2}\right)\right] = \frac{5}{4};$$

$$k'_2 = \frac{3}{2}\left[1 - \frac{1}{3}\left(1 - \frac{1}{4}\right)\right] = \frac{9}{8}; \quad k'_3 = \frac{3}{2}\left[1 - \frac{1}{3}\left(1 - \frac{1}{8}\right)\right] = \frac{17}{16}.$$

From formula (15) the fixed-end moment is calculated as follows.

$$M_A = \frac{Wl}{4}\left\{\frac{\frac{9}{8}\left(\frac{15}{4} - \frac{77}{32}\right) - \frac{1}{2} \times \frac{5}{4}\left(\frac{27}{8} - \frac{17}{8}\right)}{\left(5 \times \frac{77}{64}\right) - \frac{75}{16}}\right\} = \frac{Wl}{8} \times \frac{11}{10}.$$

By use of the foregoing formulæ, distribution factors, carry-over factors and fixed-end moments for beams with the most common forms of loading can be evaluated. Fixed-end moments for beams carrying systems of loading which have not been considered may be found by superimposing the solutions given or by developing new formulæ by the same method. Any gradual change of cross-section can be considered to the required degree of accuracy as a series of small abrupt changes.

The Yield-line Method of Analysis of Slabs.

THE following has been received from MR. ZBIROHOWSKI-KOSCIA:

"I wish to comment on two articles dealing with the method of design of slabs by the yield-line theory, namely 'Design of Slabs by the Yield-Line Method' by Mr. A. J. Ashdown, in the numbers for June and July 1960, and 'Analysis of Slabs Supported Along all Edges' by Mr. E. H. Mansfield in the number for September 1960.

In Mr. Ashdown's article the impression is given that the author sacrifices the integrity of the basic theory at the expense of simplicity; and that some of his statements and equations appear to be mathematically incorrect and not justified by the theory on which they are based. In Mr. Mansfield's article there are some results which do not seem to be in agreement with the established theory of plates.

Mr. Ashdown derives an expression for x [equation (27)] on the assumption that $\frac{dV}{dx} = 0$ for the yield-line pattern, for the maximum bending moment, m. From the Johansen's yield-line theory the external work U is equal to the internal work V. Hence introducing the function $F(\neq 0)$ such that $Fm = V$ (F is a function of the geometry of the slab and its yield-line pattern), we have

$$M = \frac{U}{F} \quad . \quad . \quad . \quad (a)$$

From this relation the yield-line pattern giving the maximum moment M can be found from the equations:

$$\frac{\delta m}{\delta x} = 0; \quad \frac{\delta m}{\delta y} = 0; \quad \text{and} \quad \frac{\delta m}{\delta z} = 0 \ . \quad (b)$$

where x, y and z are the co-ordinates of the yield-lines. From (a) and (b) and the general rules of differentiations, we have

$$\frac{\delta m}{\delta x} = \frac{1}{F^2}\left(F\frac{\delta U}{\delta x} - U\frac{\delta F}{\delta x}\right) = 0 \ . \quad (c)$$

and since $U \neq 0$ and $F \neq 0$ we have

$$F\frac{\delta U}{\delta x} = U\frac{\delta F}{\delta x} \quad . \quad . \quad . \quad (d)$$

from which the value of x can be found. Similarly from the rules of differentiation, we have

$$\frac{\delta V}{\delta x} = \frac{\delta F}{\delta x}m + \frac{\delta M}{\delta x}V \quad . \quad . \quad (e)$$

from which it follows that $\frac{\delta V}{\delta x} = 0$ only when either m (which is a maximum) or $\frac{\delta F}{\delta x}$ $\left(\text{which from d is equal to } \frac{F}{U}\cdot\frac{\delta U}{\delta x}\right)$ is equal to zero. However in this particular case, we have $\frac{F}{U}\cdot\frac{\delta U}{\delta x} \neq 0$, and consequently Mr. Ashdown's equations (27) and (28) are invalid.

The equation $\frac{dV}{dy} = 0$ happens to be true when it is used to develop Mr. Ashdown's equation (13) because in this instance $\frac{\delta U}{\delta y} = 0$. Equation (13), however, is developed also from the assumption that 'the diagonal cracks are not influenced by the work in $(a - x)$ . . .' Justification of this statement is not given in the text, and the formula for x derived by the orthodox method gives values completely different from those given by equation (13).

In the particular case when $a = 2b$ and $i_1 = i_2 = i_3 = i_4 = 1$, we have from equation (13), $X = \frac{b}{2}$, and from (15c) $m = \frac{Pb^2}{16}$; but from the equilibrium of the panels, we have for the triangular panel

$$(m_3 + m_4)b = \frac{bx^2}{6}p = 2m = \frac{P}{6}\cdot\frac{b^2}{4}$$

$$= \frac{P}{24}b^2, \text{ or } m = P\frac{b^2}{48}.$$

The results obtained by the orthodox method are $x = \frac{3}{4}b$ and $m = \frac{3}{4}\cdot\frac{b^2}{16}P$ for each panel of the slab.

Thus the bending moment based on the orthodox method differs by 25 per cent. from that determined by equation (15c). Also equation (15c) represents nothing more than the expression for the bending moments on a slab spanning in one direction and restrained so that $m(1 + 1) = q\frac{l^2}{8}$, in view of which the effort of developing the formulæ for x seems hardly necessary.

Consider the case of a square slab centrally loaded by a concentrated load P.

From equation (22) the relations, when $d \to 0$, are $\frac{P}{m} = 3{\cdot}315$ when $i = 0$, and from equation (22a) $\frac{P}{m} = 6{\cdot}63$ when $i = 1$. I believe these relations to be erroneous and, for this yield-line pattern, should be $\frac{P}{m} = 6{\cdot}63$ when $i = 0$, and 8·0 when $i = 1$.

In Mr. Mansfield's article is given the formula on the graph (*Fig.* 11) showing the relations between $\frac{Pa^2}{M}$ and the amount of top reinforcement in the uniformly-loaded square slab. The tabulated results raise doubt as to the validity of the calculations."

MR. A. J. ASHDOWN replies as follows:

"The formulæ in my article differ from Johansen's formulæ in that I have neglected the influence of the work done by the part of slab beyond the diagonal crack for, as Mr. Zbirohowski rightly says, the sake of simplicity.

Regarding equation (27), an expression for x based on equation (d) given by Mr. Zbirohowski is a formidable square-root expression. However, by making all the m-values equal, with $a = 20$ ft. and $b = 10$ ft., the resulting value of x is only 6·54 ft. compared with 10 ft. as given by equation (27); the resulting value of m is $15{\cdot}6p$ compared with $m = 16{\cdot}67p$ given by equation (28), which is only 7 per cent. greater.

With regard to equations (13) and (15c), I agree that a good case is made out for Johansen's analysis but, at the free end of a long slab, the influence of the fixed end would be very small and would be about equal to that given by equation (15c).

Mr. Zbirohowski is rather unfair in prescribing his limits for equation (22); I cannot imagine a slab with no stiffness in one direction as is assumed if $i = 0$."

Slabs supporting concentrated loads, or even line-loads, are liable to fail by shearing before the load indicated by the yield-line method is attained. This type of failure is accompanied by the appearance of a nearly circular pattern of cracks on the soffit at a radius of about $\frac{1}{2}D + 2{\cdot}6d_1$ from the centre of the load, where D is the diameter of the loaded area and d_1 is the effective depth; the factor 2·6 depends partly upon the distance to the supporting beam or wall.

MR. E. H. MANSFIELD replies as follows:

"I see that Mr. Zbirohowski's criticisms arise solely from a misunderstanding of the symbol a. In my article, as in my paper (Reference 2), the side of the square slab is $2a$, but unfortunately the symbol a is not defined in my article; all other symbols are defined in the text."

A General Equation for Frame Analysis.

THE following is an addendum to the article on "A General Equation for Frame Analysis", by Prof. A. A. L. Baker, which was given in the number for March last.

Application of Equation (2) to Continuous Beams—Precise Solutions.—If the moment-rotation curves are available for a symmetrical continuous beam bridge, say, of three symmetrically-loaded spans, there is only one unknown χ in equation (2), so that a precise solution is readily obtained. It is sometimes convenient to separate the moments for elastic conditions from those due to inelastic rotations. The latter may be found by inserting appropriate θ'-values and ϕ-values in the equations and assuming $M_0 = 0$. This method was proposed by G. Macchi using the Three-Moments Theorem to establish the equations.

As proposed by Y. Guyon, the distribution of bending moments may be obtained by an adaptation of the Hardy-Cross moment-distribution method, since the elastic deformation of frame members can be separated from the inelastic deformation, the latter being assumed to be concentrated at critical sections. This method is useful when the influence of sway is negligible.

Correction.—The term $M_1\phi_1'$ in equation (2) should read $M_1\theta_1$. The terms "$\frac{M}{\phi}$ curve" and "$\frac{M}{\psi}$ curve" relate to the curves showing M plotted against ϕ and ψ respectively.

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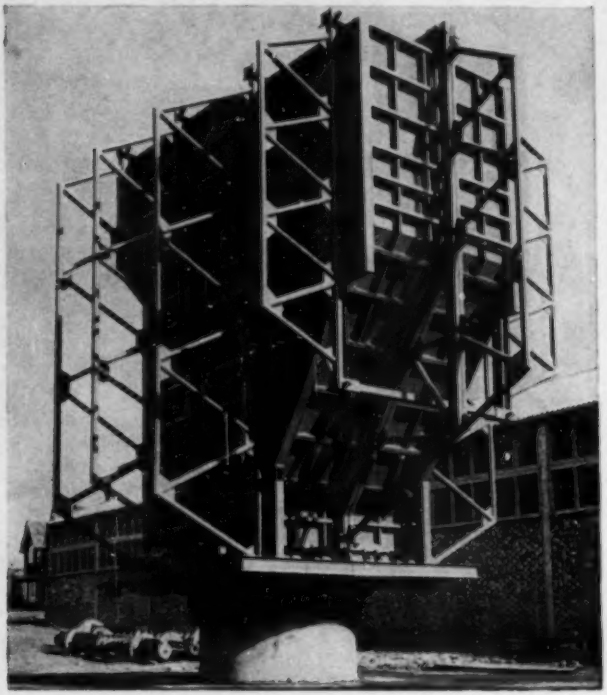

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Reinforced Cellular Concrete in Buildings.

MANY buildings have been erected recently in Great Britain, and others are in the course of construction, incorporating reinforced slabs cast in a type of cellular concrete which was developed in Sweden and has been used in Scandinavia and elsewhere for many years. The material has high thermal-insulation value and low shrinkage and moisture movement, and is incombustible. Its density is about one-fifth that of ordinary gravel concrete. Reinforced roof and floor slabs and vertical and horizontal slabs for walls and storey-height partition slabs having a width of 24 in. and lengths up to 20 ft. are made with the material. Out of a total of 1,250,000 cu. yd. of this particular type of cellular concrete which is made annually, about two-thirds is now used for reinforced slabs.

The concrete slabs are made in factories. The process is to combine a slurry of finely ground siliceous sand with ordinary Portland cement. The admixture of aluminium powder and other chemicals produces a material of cellular structure. Close control is exercised during manufacture to ensure a product of uniform quality. The mixture is cast in steel moulds which are finally placed in large autoclaves where the products are cured in steam under high pressure for about eighteen hours; this process completes the chemical reactions and ensures a stable inert product. The mild steel reinforcement, before being placed in the moulds, is treated to prevent corrosion. A high degree of mechanisation in the manufacture of the slabs ensures dimensional accuracy so that dry-construction methods of erection can be adopted.

Some examples of the use of this cellular concrete are described in the following. In the extension of a factory for Messrs. Borg-Warner, Ltd., at Letchworth, slabs 4 in. thick provide 5500 sq. yd. of flat roof. The slabs are designed to carry a live load of 15 lb. per square foot and span 10 ft. between steel beams.

Slabs, laid either vertically or horizontally, of reinforced cellular concrete provide a lightweight weatherproof walling of good thermal insulation and fire-resistant properties. The slabs are usually treated externally with a silicate preparation obtainable in various colours and which has been developed specifically for use with this type of cellular concrete.

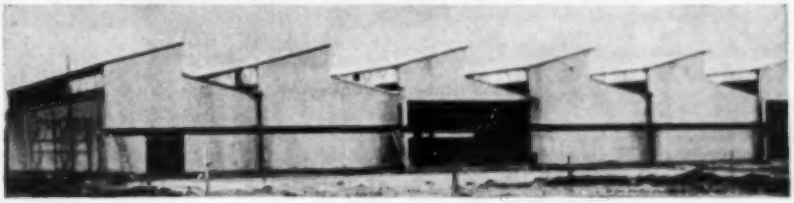

Fig. 1.—Factory Walls with Vertical Slabs.

Because of the good light-reflective and sound absorptive characteristics of the surface of the slabs, internal faces need not be painted or otherwise decorated. The use of reinforced cellular concrete slabs laid vertically to form the walling of one of the standard factories for the Cumbernauld Development Corporation is shown in *Fig.* 1. The slabs, which span between steel angle-rails from 12-ft. to 6-ft. centres, are 6 in. thick and are designed to resist wind pressure and other incidental loading.

A storage depot for Convoys Ltd., at Deptford, has a precast frame of ordinary reinforced concrete, but the roof and wall slabs are of reinforced cellular concrete. The roof slabs are 14 ft. long and 6 in. thick and bear on the flanges of the precast roof beams (*Fig.* 2). An asphalt finish was applied direct to the slabs. The horizontal slabs in the walls are 5 in. thick and span between the precast columns which are at 15-ft. centres

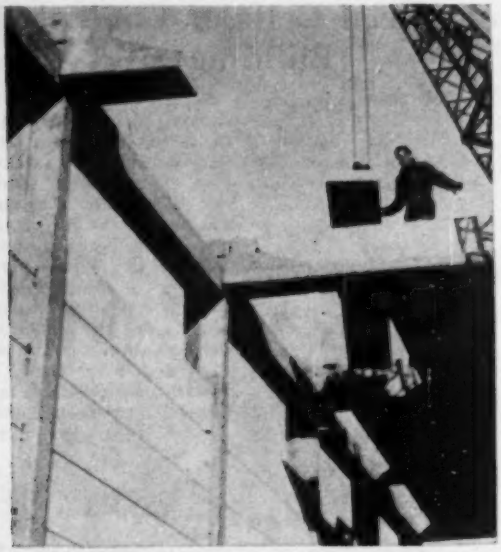

Fig. 2.—Roof Slabs in Course of Erection.

longitudinally and at 16-ft. 8-in. centres transversely. The ends of the slabs are fixed in tapered rebates in the columns. The use of lightweight roof and wall slabs in combination with a precast concrete frame resulted in rapid construction of the building.

Economy in the thermal insulation required for the roof of a building, housing "cooler rooms", for Kraft Foods Ltd., Liverpool, was effected by the use of 1800 sq. yd. of slabs of reinforced cellular concrete which for structural reasons are 4 in. thick. By taking into account the high thermal resistivity of the slabs it was necessary to provide only 4 in. of cork insulation. Another example of the use of reinforced cellular concrete proving advantageous for insulation as well as structural purposes is the roof of a bonded warehouse in Scotland, where more than 2500 sq. yd. of 6-in. roof slabs spanning 15 ft. between precast concrete pitched portal frames provided the degree of insulation necessary to maintain the internal conditions beneficial to the maturing of whisky.

An instance of a lightweight material being used to reduce the load on a supporting structure and its foundations is a store at Romford for the Times Furnishing Co., Ltd., to which has been added another storey. In the additional work, 500 sq. yd. of 6-in. reinforced cellular concrete roof slabs covered with bituminous felt are provided, and the new roof imposes an extra dead load of 20 lb. per square foot only on the structure below. In the case of a photographic instrumentation building designed by the Ministry of Works, the structure was to be as light in weight as practicable because of the poor subsoil and the fact that the building was to be erected over several large underground pipes. The provision of 5-in. floor and roof slabs of reinforced cellular concrete with the inner leaves of the walls constructed in 4-in. blocks of cellular concrete enabled this condition to be met.

The ability of lightweight reinforced cellular concrete to carry heavy loads is demonstrated in a new building for Messrs. T. Wall and Sons (Meat & Handy Foods) Ltd., at Willesden, where the roof supports, in addition to the ordinary roof loads, plant weighing the equivalent of a load of ¼ ton concentrated at the centre of each slab. The slabs are 2 ft. wide and 8 in. thick, and span 17 ft.

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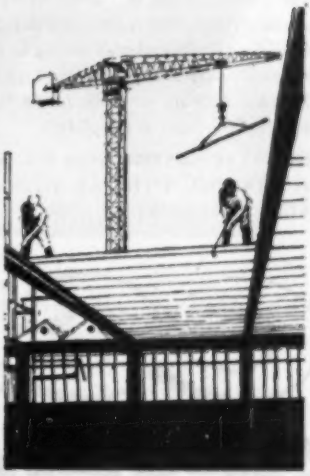

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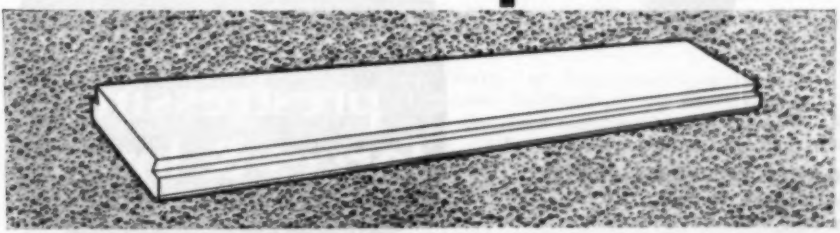

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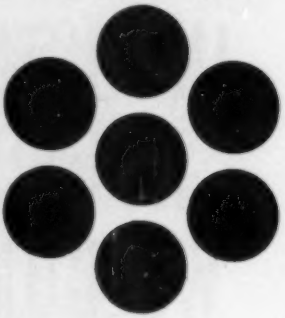
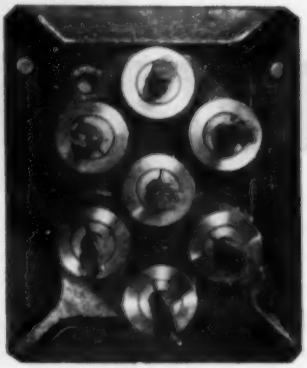
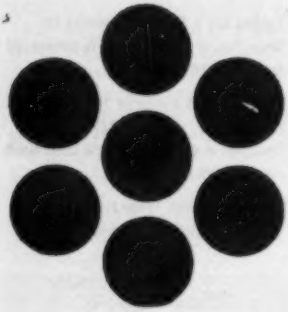


CCL

Seven Strand System





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Prestressed Concrete Division
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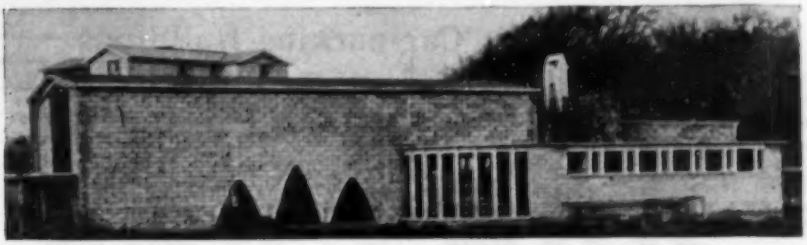

Fig. 3.—Church at Gracemount.

The church (*Fig.* 3) and church hall shown in the course of construction at Gracemount, near Edinburgh, form an ell-shaped building, the frames of which are of precast reinforced concrete and are at 9-ft. 6-in. centres. The structure comprises 5-in. roof slabs, 6-in. floor slabs, and 8-in. horizontal and vertical wall slabs, all of which are of reinforced cellular concrete. The vertical slabs are 16 ft. long.

The external faces of the wall slabs will be treated with the silicate preparation (mentioned in the foregoing) in various colours. In other parts of the structure 8-in. blocks, rendered externally, form both load-bearing and panel walls.

The factory illustrated in *Fig.* 1 is one of several designed by the Cumbernauld Development Corporation; the main contractor was Messrs. L. K. Mackenzie & Partners, Ltd.

The architects for the storage depot at Deptford (*Fig.* 2) are Messrs. Stock, Page & Stock; the precast concrete frames were designed and erected by London Ferro-Concrete Co., Ltd.

The architect for the church at Gracemount (*Fig.* 3) is Mr. Harry Taylor, A.R.I.B.A.; the contractors were Costain Concrete Co., Ltd.

The cellular concrete described in the foregoing is "Siporex" and is manufactured by Costain Concrete Co., Ltd., at works established in March 1960 at Newarthill, Lanarkshire. The information in the foregoing was contributed by Mr. A. J. Walker, B.Sc.(Eng.).

Bulletins Received.

"**Sponsored Fire-resistance Tests on Structural Elements.**" By L. A. Ashton and P. M. T. Smart. (London: H.M.S.O. 1960. 7*s.* 6*d.*)

"**Apparatus Used in the State Institute for Technical Research for Determining Thermal Conductivity.**" By R. Ruso and A. Kopsala. (Helsinki: The State Institute for Technical Research. 1960. In Finnish with a summary in English.)

"**The Temperature Field Produced in the Ground by a Heated Slab Laid Direct on Ground, and the Heat Flow from Slab to Ground.**" By O. Vuorelainen. (Helsinki: The State Institute for Technical Research. 1960. In English.)

The Temperature Changes in Concrete under Winter Concreting Conditions—Theories and Applications." By S. E. Pihlajavaara and Heikki Syrjälä. (Helsinki: The State Institute for Technical Research. 1960. In Finnish and English.)

"**Surface Cracking of Mass Concrete Structures at Early Form Removal.**" By T. C. Hansen. (Stockholm: Swedish Cement and Concrete Research Institute. Applied Studies No. 4. 1960. In Swedish with a summary in English. Price Kr. 10.)

"**Creep and Stress Relaxation of Concrete.**" By T. C. Hansen. (Stockholm: Swedish Cement and Concrete Research Institute. Proceedings NR 31. 1960. In English. Price Kr. 15.)

Design of Eccentrically-loaded Circular Columns by the Load-factor Method.

THE concluding part of the articles entitled "The Design of Eccentrically-loaded Circular Columns by the Load-factor Method", by J. D. Bennett, B.Eng., will be given in the number for May and will describe the basis of the charts published in this journal for February and March 1961.

Multiple-storey Car-parking Buildings.

In the number for January 1961, illustrations and descriptions were given of some multiple-storey car-parking buildings which are either in the course of construction or are proposed to be erected. Some further notes on some of these buildings and on other structures are given in the following.

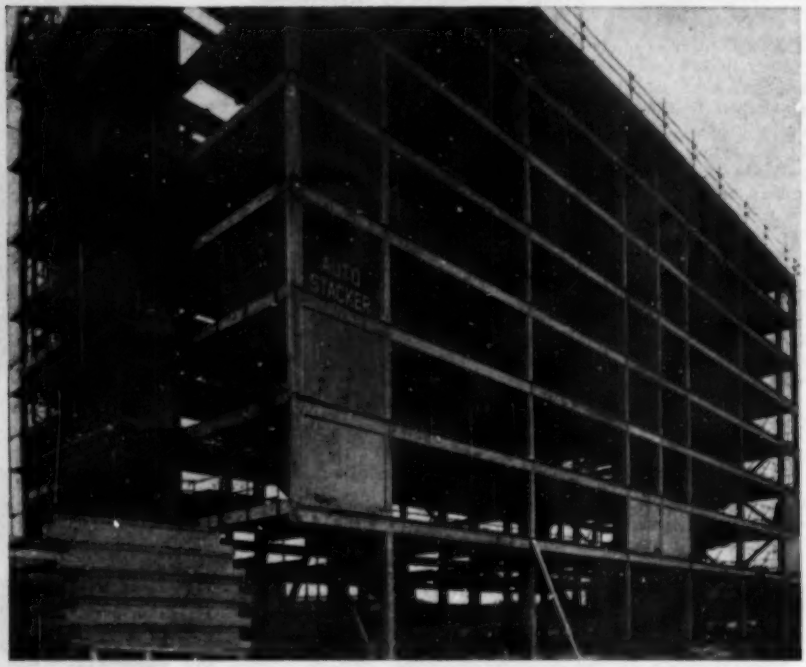

Fig. 1.

Building at Woolwich.

On page 41 of the number for January 1961, a preliminary description of a nine-storey car-parking building at Woolwich is given. This structure is illustrated in course of construction in *Fig.* 1. The building is 116 ft. by 55 ft. on plan (excluding the end staircases) and about 82 ft. high. It is founded on bored piles with the ground-floor slab acting as a tie between the heads of the piles and obviating the need for special beams. The superstructure contains more than three thousand precast concrete units, the maximum weight of a unit being 3 tons, and comprises a system of frames, cross beams and floor slabs. The units are being precast on the site with the exception of the roof purlins, which are prestressed and were made at a works in Manchester. The concrete throughout has an exposed surface straight from the moulds. The use of a limestone aggregate in the concrete for the frames and beams enables the minimum size of column to be 9 in. while maintaining the requirements of the London By-laws for fire resistance.

There are four lines of the frame units running the length of the building on plan and defining the three bays into which the structure is divided. The frame units are tied at each floor with the cross beams and floor slabs which transmit the wind forces on the longer elevation to a number of

MULTI-STOREY CAR PARK CONSTRUCTION

For the City of Cardiff. Architect: Mr John Dryburgh, City Architect.

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inclined braces. There are three groups of braces in each of the outer bays, one in the central bay and one near each end. The bracing forms a V between the exterior and interior columns of the frames at the first, third, fifth and seventh floors, and an inverted V at the ground, second, fourth and sixth floors. There is no bracing at the eighth floor.

The structure will have no wall cladding. The 5-in. precast concrete panels forming the end walls of the central bay, through which the cantilevered stairs are bolted, are required for fire resistance, and the provision of 4-in. infilling panels above the lift entrances on the longer elevation allow the corner and central columns to be omitted so as to give easier access to the lifts.

The frame units are of a system called the "picture frame" system, but, in this case the bottom member being omitted, each frame comprises a top beam and two half columns. The sizes of the units vary according to their positions in the structure. The lengths of the top beams vary from 8 ft. 10 in. to 20 ft., and the height of the columns from 7 ft. 5 in. to 12 ft. 4 in. The columns for the longer spans are 12 in. by 4½ in. and for the shorter spans 10 in. by 5 in. The ell-beams are from 8½ in. to 12 in. deep and have a maximum width at the bottom of the ell of from 14 in. to 16 in. The perimeter frames for the upper storeys have tubular metal guard-rails cast in them which serve the purpose of providing stiffness during erection and a safety rail when the building is in service. The units are bolted together and subsequently pressure-grouted.

The floors comprise 1½-in. precast reinforced concrete slabs covered with a 1½-in. cast-insitu structural screed of reinforced concrete.

The architects to Woolwich Borough Council are Messrs. T. and P. H. Braddock. The design and construction of the building and precasting of the units including the prestressed concrete purlins is undertaken by Truscon Ltd., on behalf of Auto-Stackers Ltd. The sub-contractors for the bored piles were Frankipile Ltd. The mechanical installation is being carried out by Ace Machinery Ltd.

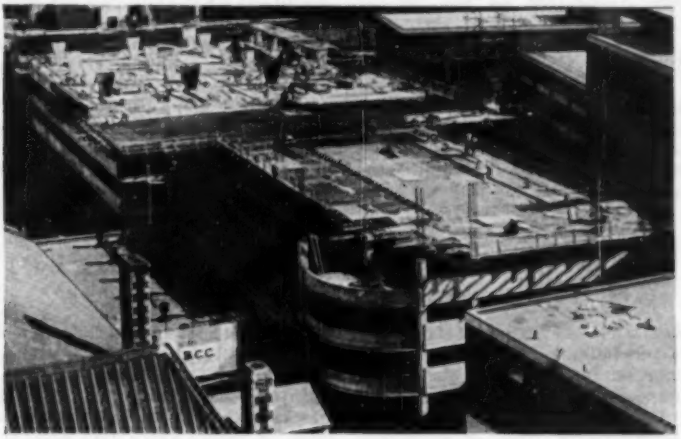

Fig. 2.

Building at Cardiff.

In *Figs.* 2 and 3 is illustrated a multiple-storey car-parking building being erected by Building & Construction Co. Ltd., for the City of Cardiff, the architect being Mr. John Dryburgh, the City Architect.

Building at Birmingham.

In the March number reference is made to a multiple-storey car-parking building to be erected for the Austin Motor Co.

Fig. 3.

at Longbridge, Birmingham. A feature of this structure is that the inclined ramps are being constructed on the lift-slab principle.

Buildings at Bristol, Bath, and Leicester.

Car-parking buildings erected for Multidek Car Park Developments at Bristol, Bath, and Leicester were described and illustrated on pages 39 and 40 of the number of this journal for January 1961. The architect for the buildings at Bristol and Bath is Mr. R. Jelinek-Karl, and the architects for the building at Leicester are Messrs. Fitzroy, Robinson & Partners. The consulting engineers for the building at Bristol are Messrs. G. C. Mander & Partners in association with Mr. E. N. Underwood, and for the buildings at Bath and Leicester, Messrs. G. C. Mander & Partners alone. The contractors for the three buildings are Messrs. William Cowlin & Son, Ltd.

Buildings for an Architectural Congress.

THE illustration on page 139 shows the headquarters building (in the foreground), the exhibition building, and the tower, which have been designed by Mr. Theo Crosby, A.R.I.B.A., for the sixth Congress of the International Union of Architects which is to be held in London in July 1961. (See Editorial Note on page 139.) The headquarters building is mainly of glass and aluminium with " Asbestolux " partitions, and is being sponsored by Messrs. Pilkington Brothers, Ltd., The British Aluminium Co., Ltd., and Cape Building Products Ltd.; it will be built at cost by Taylor Woodrow Construction Ltd., which firm are also to erect the exhibition building free of cost to the organisers.

Among the relatively new materials which will be used in the construction of the exhibition buildings will be " Spectra-glaze ". This is a structural concrete building block with a face of coloured glazed finish made by a process employed in Britain by the Atlas Stone Co., Ltd. The glaze is cast on to standard concrete blocks and consists of a thermo-setting resinous binder which runs into the pores of the concrete, thus becoming an integral part of the block. It is claimed that this building component block combines structural strength with a permanent finish in a wide range of colours, and that building and finishing is carried out in one operation, thereby saving time and cost. Two features in " Spectra-glaze " are to be provided in the exhibition building, namely, a circular wall 28 ft. in diameter enclosing the book exhibition, and the wall which is to enclose the central control enquiry section. The Atlas Stone Co., Ltd., are also to provide the textured-surface concrete paving in the main exhibition hall.

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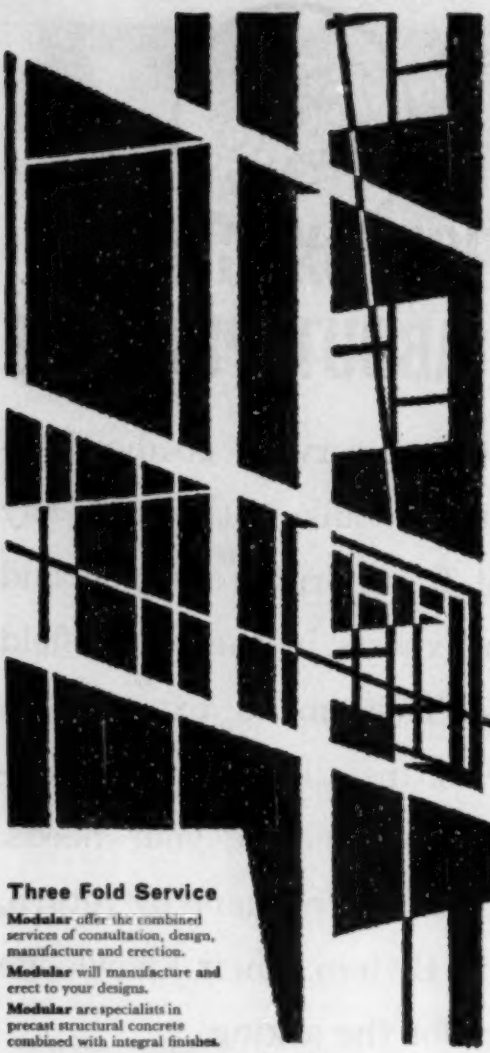
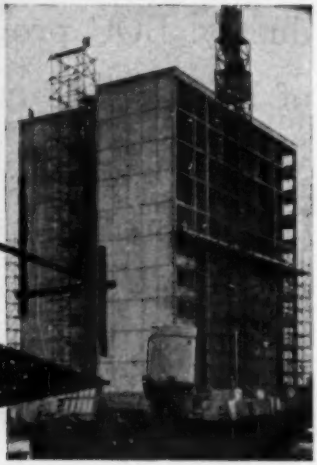


Pattern of service in the concrete age



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Glass-fibre Shuttering for a Helical Stair.

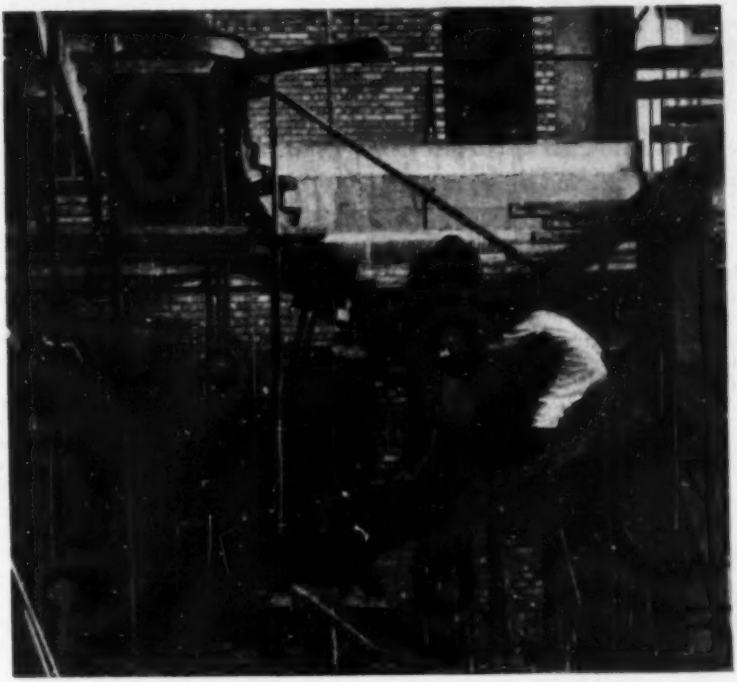

Fig. 1.

At the new premises of the North Thames Gas Board at Fulham, London, some constructional features of which are described in the number of this journal for October 1960, a helical stair has been constructed, using a glass-fibre mould (*Figs.* 1 and 2). The stair is 16 ft. in height and, in plan, is in the form of two semicircles with a straight portion between. It comprises thirty-two treads and landings at the mid-height and at the top. The mould was in one piece and was made at a works a few miles from the site. After erecting the mould in position the steel reinforcement was fixed and the concrete placed insitu. It is claimed that this method of construction is a radical departure from the usual method of providing timber shuttering constructed with an

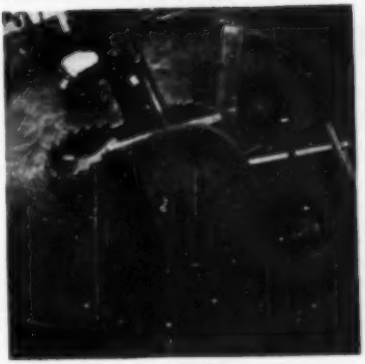

Fig. 2.

inner and outer drum, filling and developing the soffit, cutting away for the treads and risers, and dismantling and scrapping the shuttering after use. With the glass-fibre mould time is saved and, when the concrete has hardened, the mould can be removed for re-use and leaves a smooth and true surface.

The architects are Messrs. E. R. Collister and Associates, and the general contractors are Messrs. Trollope & Colls Ltd. The sub-contractors are Pierhead Ltd., and the mould was made by D.A. Models Ltd.

Messrs. Christiani & Nielsen Ltd.

It is regretted that in the advertisement for Messrs. Christiani & Nielsen Ltd., civil engineers and contractors, on page lxiii of the number for March last, the name of the firm is spelt incorrectly.

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Bursaries in Concrete Structures and Technology

Bursaries of £460–£760 according to experience are available for Session 1961–1962. Candidates must hold a degree in engineering and have a good knowledge of the theory of structures. Full information from the Registrar, Imperial College, London, S.W.7. Closing date 1st June, 1961.

THE UNIVERSITY OF LEEDS

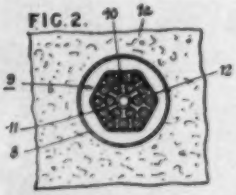
The University offers a one-year full-time course for a Diploma in Concrete Technology commencing in October, 1961. The course consists of lectures, laboratory work, and a design or research project.

Applications for admission to the course are invited from graduates and from holders of equivalent qualifications in engineering. Preference will be given to applicants who have had one or two years' practical experience.

Further information may be obtained from The Registrar, The University, Leeds, 2.

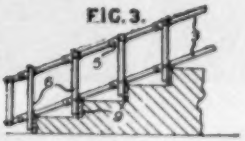
Patent Applications.

Reinforced Concrete.



A tendon, for reinforcing concrete, comprises a plurality of steel wires 10 in a wrapping 12, the spaces between the wires being filled with a hardenable material, e.g. cement or synthetic resin, in its unhardened condition. A spiral member 11 may be incorporated for the injection of hardening fluid either before or after tensioning. Heat can be applied by passing an electric current down the wires if synthetic resin is used. The space between the member 8 and the tendon is filled with concrete.—No. 838,090. Span Tendons Ltd. September 22, 1958.

Shuttering for Stairs.



Angularly adjustable shuttering for casting a stairway comprises shutter boards 9 for the risers supported on each side by hangers 6 adjustably mounted on parallel rails 3. The hangers 6 are pivoted on sleeves 5 slidable along the rails so that the hangers can be adjusted and locked by clamp screws in the correct position and the shuttering then adjusted to the correct angle of inclination in which it is supported while the stairway is cast. While the shoes are adjusted, the frame 3, 6 is locked by a diagonal stay. The shuttering is supported by walls or on an upright at the higher end. The shuttering is completed by boards at the sides of the boards 9 if no wall serves the purpose. —No. 836,244. J. Shaw. May 23, 1958.

Cement for the Forth Road Bridge.

THE illustration shows the 200-ton elevated silo at the South Leith depot of British Railways from which, as described previously,* some of the cement is supplied in pressurised-tank road vehicles to the works in connection with the new road bridge over the Firth of Forth. Some of the cement, which is for the construction on the southern shore of the Firth, is discharged directly from the train of 20-ton pressurised-tank wagons seen on the left-hand side of the silo.

* See this journal for March 1961.

Approach Works for the Forth Road Bridge.

THE Forth Road Bridge Joint Board has awarded a contract for £2,250,000 for the northern approach road and another for £920,000 for the construction of the approach viaducts. The former contract will provide a motorway 3½ miles long in Fife. The northern viaduct will be 275 yd. long and will comprise six spans, and lead into two tunnels 1000 ft. long through the rock of St. Margaret's Head. Ten bridges are to be constructed along other parts of approach roads and four miles of connecting roads.

DATA FOR PRICING REINFORCED CONCRETE.

MATERIALS (*Delivered London*).

AGGREGATES (per cu. yd.).
Washed sand: 26*s.* 9*d.* Graded gravel (¾ in.): 24*s.* 3*d.* Pit ballast: 25*s.* 6*d.*

CEMENT (per ton).
Ordinary Portland.—6 tons and upwards: 112*s.*; 1 ton to 6 tons: 124*s.*
Rapid-hardening Portland: As ordinary Portland plus 10*s.* 6*d.* "Aquacrete" and "417": As ordinary Portland plus 32*s.* 6*d.* "Colorcrete" (buff, red, and khaki): As ordinary Portland plus 60*s.* "Snowcrete" (white): 277*s.* 6*d.*
"Super-Cement": As ordinary Portland plus 32*s.* 6*d.*
High-alumina cement.—1 ton: 337*s.*; 2 tons: 327*s.* 6*d.*; 6 tons: 321*s.* 6*d.*
"Super Snowcem" waterproof cement paint: 100*s.* per cwt.

REINFORCEMENT (per ton ex stock including delivery in quantities from 10 to 500 tons).
Plain mild steel rods per B.S. No. 785.—1 in. and over: £50 5*s.*; ⅞ in. and ¾ in.: £51; ⅝ in.: £51 15*s.*; ½ in.: £53 7*s.* 6*d.*; ⅜ in.: £54 17*s.* 6*d.*; ¼ in.: £58 17*s.* 6*d.*
Hot-rolled deformed bars: As plain bars plus £1.
High-tensile (minimum yield point 44,000 lb. per sq. in.): As mild steel plus 19*s.*
(For basic prices and allowances for quantities exceeding 500 tons and extras for quantities less than 10 tons, lengths greater than 40 ft. or less than 5 ft., bundling, labelling, and delivery in other areas, see "Concrete Year Book, 1961".)
Fabric per B.S. 1221A (per sq. yd.).—No. 109: 3*s.* ½*d.*; No. 121: 5*s.* 7½*d.*; No. 124: 3*s.* 4½*d.*; No. 125: 2*s.* 10*d.*; No. 126: 2*s.* 6½*d.*

TIMBER.—Sawn deal shutter boards: £95 to £100 per standard (165 cu. ft.); 1-in. boards: 1*s.* (av.) per sq. ft.; 1½-in. boards: 1*s.* 3½*d.* (av.) per sq. ft.

MATERIALS AND LABOUR.

Based on net cost of materials plus 10 per cent. and net cost of labour plus 45 per cent. for insurances, etc. (10 per cent.), on-costs (22 per cent.) and profit (10 per cent. on gross cost = 13 per cent. on net cost). No allowance for non-productive ordinary time, overtime, travelling time, fares, subsistence, etc. Wages (per hour).—Carpenters and joiners: 5*s.* 1½*d.*; labourers: 4*s.* 6*d.*; mixer and hoist operators: 5*s.*; bar benders: 4*s.* 11½*d.*

PORTLAND CEMENT CONCRETE (1 : 2 : 4).
Foundations (per cu. ft.).—3*s.* 6*d.*
Suspended slabs (per sq. yd.).—6 in., 18*s.*; 7 in., 21*s.*; 9 in., 27*s.* 2*d.*
Beams (per cu. ft.).—4*s.* Columns (per cu. ft.).—4*s.* 6*d.*
Walls (per sq. yd.).—6 in., 19*s.* 9*d.*; 9 in., 29*s.* 6*d.*;
12 in. and over (per cu. ft.), 3*s.* 6*d.*
Extra (per cu. yd.) for rapid-hardening Portland cement. 1 : 2 : 4.—2*s.* 8*d.*

REINFORCEMENT.—Mild steel rods cut, bent and fixed (per cwt.).

	In floor slabs and beams	In columns	In walls.	Stirrups, binders, etc.
1 in. and over	74*s.* 9*d.*	80*s.* 5*d.*	78*s.* 1*d.*	—
⅞ in. and ¾ in.	75*s.* 7*d.*	81*s.* 3*d.*	78*s.* 11*d.*	—
⅝ in.	76*s.* 5*d.*	82*s.* 1*d.*	79*s.* 9*d.*	—
½ in.	83*s.* 11*d.*	90*s.* 8½*d.*	88*s.* 6*d.*	—
⅜ in.	85*s.* 6*d.*	92*s.* 3*d.*	90*s.* 1*d.*	108*s.* 7*d.*
5/16 in.	87*s.* 9*d.*	94*s.* 6*d.*	92*s.* 4*d.*	104*s.* 10*d.*
¼ in.	88*s.* 10*d.*	95*s.* 8*d.*	93*s.* 5*d.*	105*s.* 11*d.*

Fabric per B.S. No. 1221A in flat areas (per sq. yd.) including straight cutting.—No. 109: 4*s.* 4*d.*; No. 121: 7*s.* 11*d.*; No. 124: 4*s.* 9*d.*; No. 125: 4*s.* 1½*d.*; No. 126: 3*s.* 6½*d.*

SHUTTERING (FORMWORK).—Assuming five uses of wooden shutters for floor soffits and walls, and three uses for beams.
Soffits of floors, flat roofs, etc. (per sq. yd.): 16*s.* 3*d.*; ditto sloping: 18*s.* 1*d.*
Walls and partitions (per sq. yd.): 19*s.* 11*d.*; ditto curved: 26*s.* 10*d.*
Rectangular beams, lintels, columns, etc. (per sq. ft.): 2*s.* 8*d.*; ditto circular: 3*s.* 8*d.*

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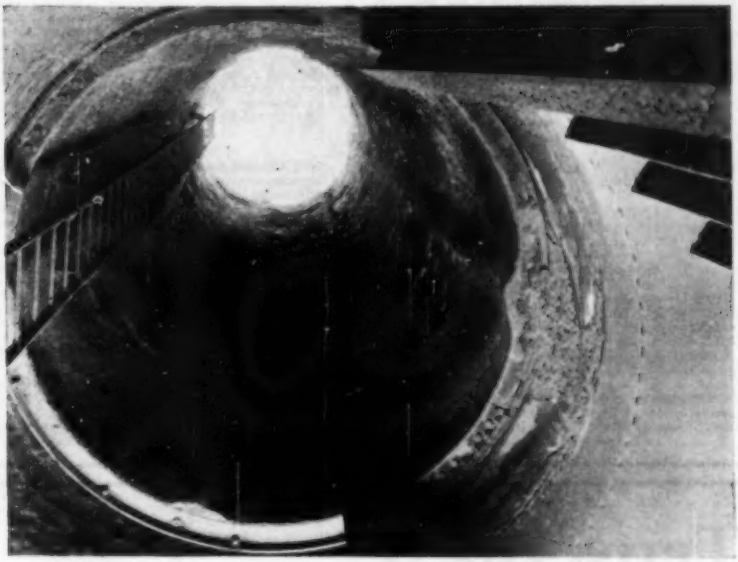
CHIMNEY LININGS THAT LAST



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Chimney stacks lined with steel, Portland cement concrete or ordinary brickwork mortar are seriously affected by corrosion caused by heat, condensation, acidity or sulphurous combustion gases. To prevent such deterioration, it is now standard practice to line chimneys with Ciment Fondu mortar, because of its high resistance to dilute sulphur acids and sulphates, and its ability to provide a strong bond even at a temperature well above that of the hottest stack.

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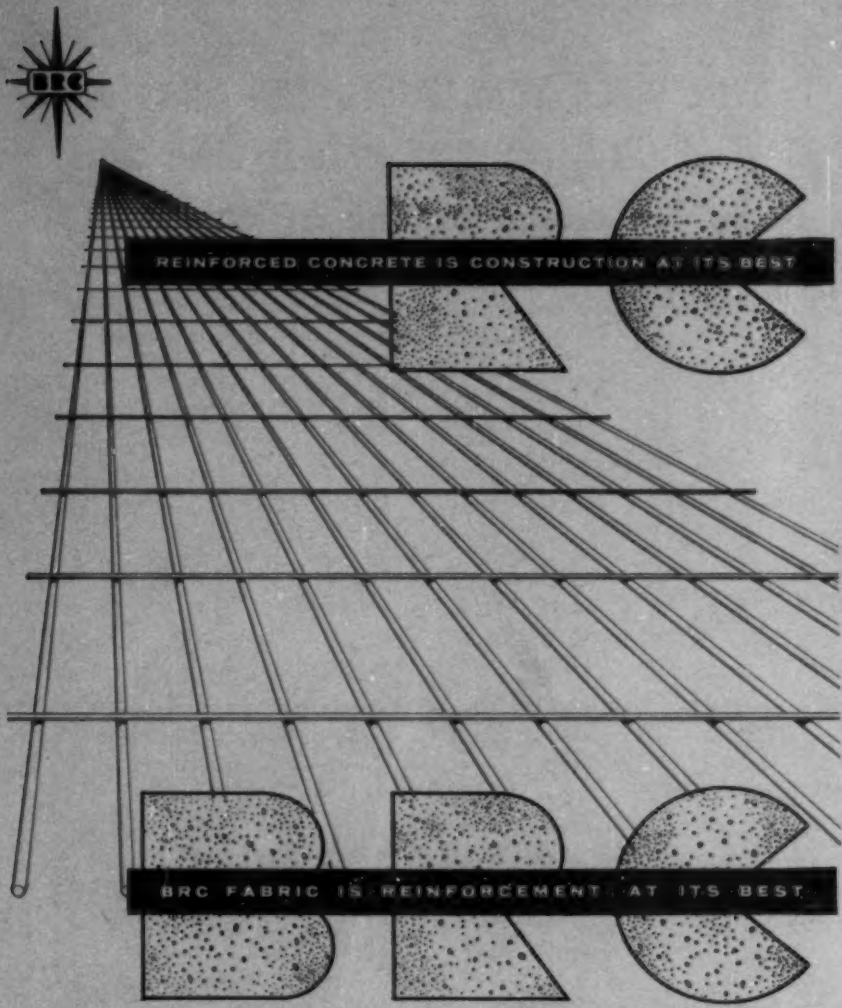

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